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PACKERY CHANNEL FEASIBILITY STUDY: INLET FUNCTIONAL DESIGN AND SAND MANAGEMENT

REPORT 1 OF A TWO-PART SERIES

Final Report

by

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Preface

The present report is Report 1 of a two-part series investigating the feasibility of the proposed Packery Channel. Report 1 covers inlet functional design and sand management. Report 2 (Brown and Militello 1997) covers bay circulation and water level. These reports were prepared under contract with Naismith Engineering, Inc., as part of a cooperative effort involving several studies approved by the Nueces County Commissioners Court, Texas. The work described in this report was authorized by Naismith Engineering, Inc., on February 12, 1996, and conducted at the Conrad Blucher Institute for Surveying and Science, Texas A&M University-Corpus Christi by Dr. Nicholas C. Kraus, Director of the Division of Coastal and Estuarine Processes, and by Daniel J. Heilman, Research Scientist. Mr. John Michael, Vice President, Naismith Engineering, Inc., was the contract technical monitor. The work for this study began in February, 1996, and was completed in February, 1997. The first draft of this report was released on September 30, 1996, and subsequent revisions of the draft were released on November 22, 1996, and December 10, 1996, with the present version *mutatis mutandis*.

Several Blucher Institute staff members assisted in this study. Dr. Ping Wang, post-doctoral researcher in the Department of Geology, University of South Florida, and Guest Researcher at the Blucher Institute assisted in the beach profile survey and in leading the wind-blown sand transport field data collection. Dr. Wang wrote Appendix F together with Dr. Kraus and Mr. Heilman. Ms. Anne-Lise Lindquist, Research Scientist at the Blucher Institute, prepared Appendix G, the storm compilation. Ms. Deidre Williams, graduate student, and Ms. Kendall Keyes and Ms. Margie Langley, undergraduate students, prepared several of the figures, including the site location map; Mr. Daryl Slocum, Head of the Blucher Environmental Instrumentation and Calibration Laboratory, was the boat operator for the sea sled survey; and T. Zachariah Jeffries, Greg Hauger, James Rizzo, and Gerardo Arrambide assisted in establishing the survey benchmarks and survey control. We thank Mr. Michael; Mr. James Shiner and Dr. Joe Moseley of Shiner, Moseley, and Associates, Inc.; and Mr. James Goldston, Goldston Engineering, Inc., for providing background information. We also thank Dr. Mark R. Byrnes, Coastal Studies Institute, Louisiana State University, who assisted in calculation of shoreline change from aerial photography.

Executive Summary

The proposed Packery Channel would reopen the historic inlet to the Gulf for Corpus Christi Bay. Because the channel would be located in a region of low net longshore sand transport and in the southeast corner of the bay, it would be well situated for stability. The following is a summary of the recommended basic functional design and accompanying practices that were arrived at in this study. The recommendations are made based on review of the literature of the physical processes at the site, field data collection, numerical modeling of shoreline change and inlet hydrodynamics, comparison with neighboring inlets, and records of the behavior of inlets around the coast of the United States.

1. The jetties should be impermeable and extend at least 1,400 ft from the position of the MSL shoreline. A 12-deg orientation north from shore-normal will provide sheltering from southeast summer waves. Outside spurs near the ends of the jetties will reduce transport of sediment into the channel by redirecting the material away from the entrance. The spurs will also reduce possible recession of the shoreline adjacent to the jetties.
2. The design channel depth is 11 ft MSL (10 ft MLLW), with a 140-ft bottom width, 1-on-3 channel side slopes, and 300 ft between jetties.
3. The deposition basin east of the SH361 bridge and the inlet entrance should be over-dredged to reduce long-term maintenance cost by reduction of dredging frequency. Over-dredging at the Gulf entrance to 13-ft MSL or greater is desirable and expected because of draft requirements of ocean-going dredges.
4. Minimization of wind-blown sand intrusion into the channel is essential and is a cost-effective means of reducing maintenance dredging volume.
5. Maintenance dredging will be an integral part of the long-term cost of Packery Channel. The estimated average annual dredging volume is 100,000 ± 50,000 cu yd. The sand dredged from the entrance channel and from the deposition basin should be placed on the downdrift beach.
6. The ebb current in the inlet entrance will steepen waves, as is the case at all tidal inlets. For waves of 5-sec period, breaking in the channel for a stronger ebb current is expected only for incident waves with heights greater than about 6 ft. Because these breaking waves will prohibit safe navigation, small-craft advisory warnings are expected to discourage boating during this sea state.
7. The adjacent beaches should be monitored through periodic beach profile surveys and aerial photography to document the impact of the jetties and to compensate through mechanical sand bypassing. Mechanical bypassing probably will not be necessary during occasional short-term (1- to 2-year) net transport reversals to the north, depending on wave climate.

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1. Introduction

Packery Channel is the historic southeast corner inlet for Corpus Christi Bay as discussed by Collier and Hedgpeth (1950) and Price (1952). This inlet connecting Corpus Christi Bay and the Gulf of Mexico closed sometime after dredging in 1912 of a 12-ft deep boat channel from the Aransas Pass inlet into Corpus Christi Bay (Price 1947), and it has remained closed (except for temporary opening by storms) since the late 1930s and stabilization of Aransas Pass and deepening of the Corpus Christi Ship Channel (presently maintained to 47-ft depth). The present study was performed to consider the feasibility of re-opening and maintaining Packery Channel.

Problem Statement

As one of two Quality of Life projects, Nueces County, Texas, is considering re-opening and permanently maintaining Packery Channel, located at the junction of the once-separated Padre and Mustang Islands. In the past and more recently, interest has been expressed in re-opening Packery Channel because it is expected to provide a number of benefits: (1) coastal recreational amenities to the public; (2) small-boat access to the Gulf of Mexico; (3) water exchange between the Gulf of Mexico and the Corpus Christi Bay-Upper Laguna Madre system; and (4) a migration route for marine organisms. The project permit is given in Appendix A.

Certain technical issues have been identified about the feasibility of maintaining the pass, including the required maintenance costs, erosion of the adjacent Gulf shoreline and channel banks, and environmental consequences of re-opening the pass. Numerous studies exist of both natural and dredged passes in the area which address many of these issues. Principal studies are Lockwood, Andrews & Newnam (1959), Turner, Collie & Braden, Inc. (1967a, 1967b, 1970), Watson and Behrens (1976), Behrens *et al.* (1977), Goldston Engineering, Inc. (1985a, 1985b), Duke (1985a, 1985b), Shiner, Moseley and Associates, Inc. (1987), Shiner, Moseley and Associates, Inc., and Duke (1989), and Morton (1989). However, extensive new data and resources are now available for the study area, including the Texas Coastal Ocean Observation Network (TCOON) of water-level and wind measurement stations and current meters located at the JFK causeway, to more reliably re-assess the feasibility of re-opening the channel. The present report is Report 1 of a two-part series investigating the feasibility of the proposed Packery Channel. This study is part of a coastal processes assessment conducted by the Conrad Blucher Institute for Surveying and Science at Texas A&M University-Corpus Christi in support of the plans of Nueces County. Report 1 addresses inlet stability, shoreline processes, and sand management issues associated with the permanent re-opening of Packery Channel. Report 2 (Brown and Militello 1997) addresses bay circulation and water level associated with re-opening of the channel.

Background and Overview of the Site

Packery Channel is a closed natural tidal inlet which serves, together with nearby Newport Pass and Corpus Christi Pass, as part of a complex of storm-washover channels. The study area is located in the City of Corpus Christi, Nueces County, Texas, at the southern terminus of Mustang Island (Figure 1). Mustang Island is a wave-dominated, high-profile depositional sandy (fine-grained sand) barrier island with a northeast-southwest orientation and extends approximately 20 miles between Padre Island (to the south) and Aransas Pass (to the north). Beaches in the area are broad, and the surf zone consists of a gently sloping forebeach and multiple-bar system. Because of its close location to the Kennedy Causeway, which connects the island to inland Corpus Christi, the Packery Channel area has become a popular tourist destination and the site of considerable resort development. A 12-ft high, 4,200-ft long concrete seawall on north Padre Island is located between two former openings of Packery Channel. This seawall, built in 1967 (Morton 1988), protects hotels, condominiums, and a small resort community located just southwest of the proposed inlet.

Together with nearby Corpus Christi and Newport passes, Packery Channel has functioned intermittently as a natural tidal inlet to Corpus Christi Bay and has been periodically reopened by hurricanes. Packery Channel is the oldest of the three passes and existed as late as the 1920s in a deeper and more stable state than was ever achieved by either of the two newer (1933) passes (Collier and Hedgpeth 1950). Based on an 1887 chart of the study area, Packery Channel (originally known as Corpus Christi Pass) served as the natural tidal inlet for Corpus Christi Bay (Morton 1988) with a depth of around 10 ft mean low water (U.S. Coast and Geodetic Survey 1887). Since 1932, the dredging of the ship channel through Corpus Christi Bay and the maintenance of Aransas Pass to depths of 47 ft mean low tide (at present) (U.S. Army Corps of Engineers (USACE) 1992) have contributed to the reduction of hydrostatic head and the gradual shoaling of the three passes (Collier and Hedgpeth 1950, Price 1952). Presently, the shortest route from the Upper Laguna Madre-Corpus Christi Bay system to the open Gulf of Mexico is via a 23-mile run along the Gulf Intracoastal Waterway (GIWW) and Corpus Christi Ship Channel to the jettied Aransas Pass. Opening Packery Channel would reduce this trip to 3 miles (Shiner, Moseley and Associates, Inc. 1987). Previous attempts at creating a permanent pass through Mustang Island include an effort in 1893 by Col. Elihu Ropes, who dredged a channel approximately 5 miles northeast of Packery Channel (Ellis 1996), efforts in 1928 and 1938 at Corpus Christi Pass (Collier and Hedgpeth 1950, USACE 1994), and an effort in 1972 by the Texas Parks and Wildlife Department, which dredged a fish pass (Watson and Behrens 1976, Behrens *et al.* 1977) near the location originally selected by Col. Ropes. Figure 2 shows the existing condition of the proposed project site.

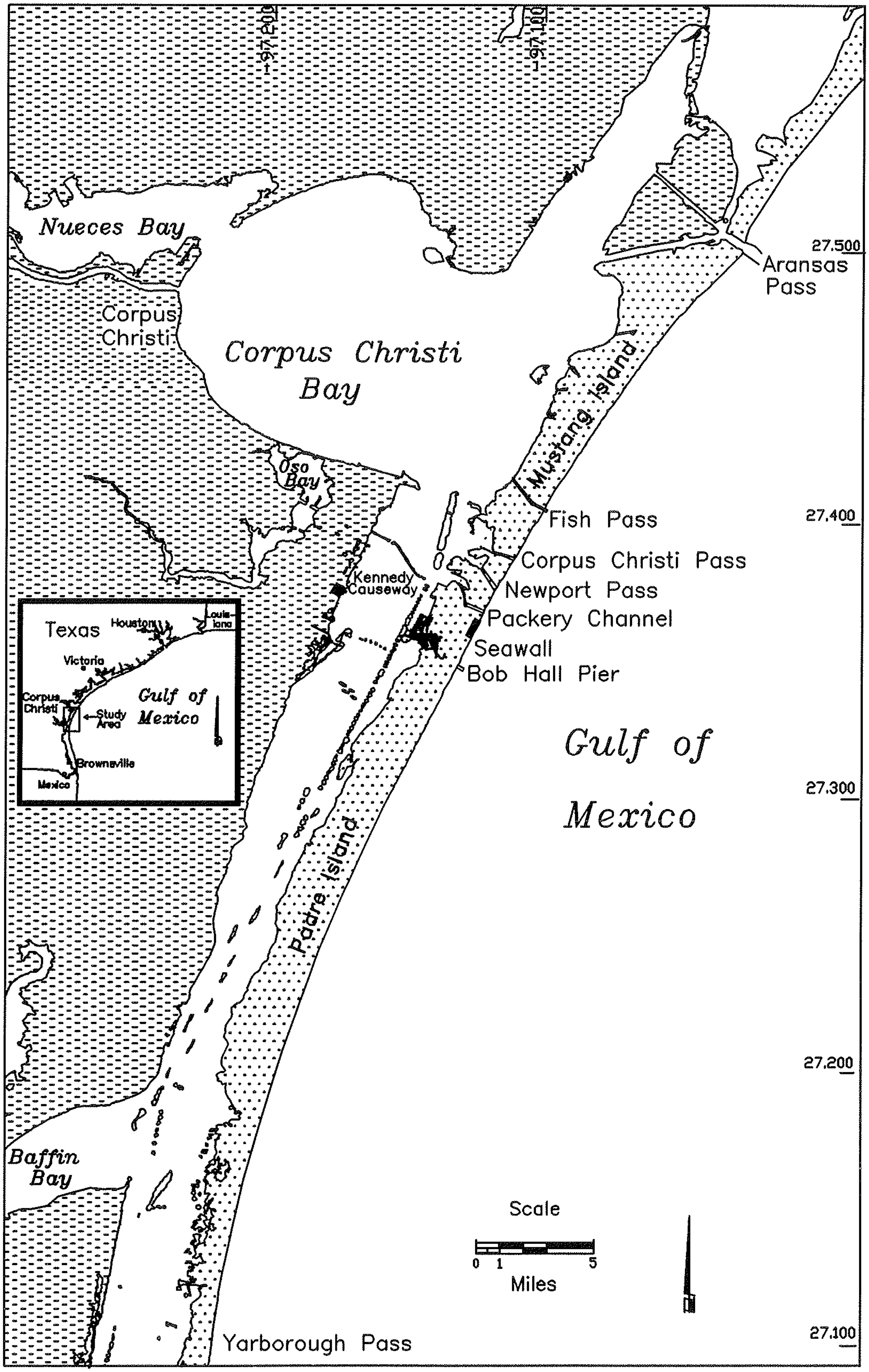

Figure 1. Site location map.

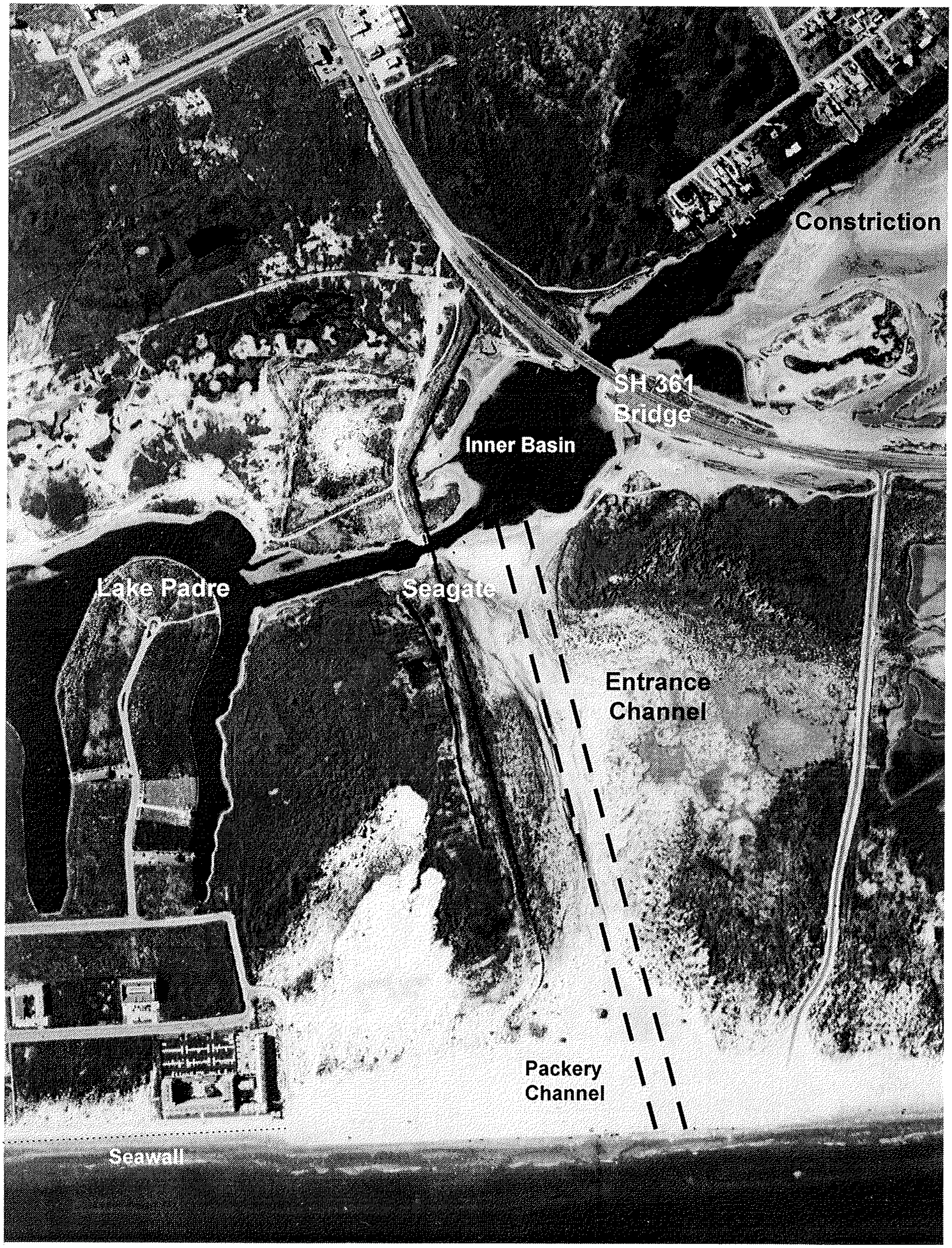

Figure 1. Aerial photograph of Packery Channel, January 12, 1996 (Lanmon Aerial Photography, Inc.).

Objectives of the Study

This report is concerned with two applications of coastal processes, inlet entrance stability and design, and sand management. A companion report (Brown and Militello 1997) treats bay circulation and water level.

The main issues in design of an inlet are stability and navigation safety. Stability refers to the tendency of an inlet to remain open by self-scouring or removal of sediments that are deposited in it, such as those brought by the longshore current or by wind. The tendency for stability will determine the amount of maintenance dredging that is required to assure design depths and safe navigation. A relatively strong ebb current is typically required for inlet stability on the Texas coast (Price 1952). On the other hand, a strong ebb current may pose a threat to navigation, because waves approaching an opposing current will steepen[1] and, possibly, break. Inlet design must balance stability and navigation, typically placing limits on channel width or the width between jetties and on the location and orientation of the jetties.

The main issue in sand management is the interruption of the littoral drift by the inlet jetties, possible creation of an ebb-tidal shoal, and resultant impacts on the shoreline, which may advance seaward on the up-drift side and recede on the down-drift side. Optimal placement of beach-quality material on the beach that is dredged from the channel during new-work and maintenance operations, as well as mechanical bypassing of littoral material that is blocked by the updrift jetty, are also elements of the sand-management plan. Inlet design and sand management considerations are therefore linked through interruption of the littoral drift and the inlet dredging (including mechanical bypassing) schedule.

The purpose of this study is to identify and quantify key coastal processes and to make recommendations for geometric parameters governing the inlet entrance design and sand management for the proposed Packery Channel. Main parameters and conditions investigated are:

1. Optimum jetty alignment.
2. Minimum length of the jetties (from the present location of the shoreline).
3. Design depth of the channel.
4. Expected average annual volume of material to be dredged and bypassed.

[1] Wave steepening refers to an increase in the ratio of wave height to wavelength. For waves riding on an opposing current (ebb current), the wave height will increase and wavelength will decrease as compared to a wave moving in the same depth of water but without a current. Similarly, waves riding on a following current (flood current), the wave height will decrease and the wavelength will increase (see page 89).

5. Impacts of the jetties on the existing shoreline and means to mitigate the impacts through jetty design and management of the new-work and maintenance dredged material.
6. Expected stability of the inlet and implications for navigation safety.
7. Implications of opening of Packery Channel on the Corpus Christi Ship Channel.

Some information is also provided on expected changes in circulation and water level in Corpus Christi Bay and the Upper Laguna Madre.

Scope of Report

Chapter 1 of this report provides the background for the study. Chapter 2 gives a detailed description of the site, including the data acquired as part of this study, and covers the acting physical processes of the area, dredging history of Mansfield Pass (a nearby maintained channel which is similar to the proposed channel), and a history of the Mustang Island Fish Pass. Chapter 3 describes the sediment budget, the functional jetty design, and associated sand management (as developed through shoreline change numerical modeling). Chapter 4 includes an analytic model of inlet hydrodynamics and an analysis of inlet stability and dredging requirements. Chapter 5 contains conclusions and recommendations. Appendices contain summaries or listings of the data collected, generated, and analyzed as part of this study.

2. Previous Studies and Existing Condition

This chapter describes previous studies and present physical condition at the project area. To supplement available information on the site, project-specific studies were conducted during April and May, 1996, to survey the beach profile, sample the sediment on the profile, obtain a wave hindcast, and quantify the wind-blown sand transport rate.

Geology and Geomorphology

The project area is located at the southernmost end of Mustang Island where, since the gradual shoaling of Packery Channel (originally named Corpus Christi Pass) in the 1930s, the island has merged with Padre Island to the southwest (Figure 3). The site is near the middle of the 290 miles of extensive barrier islands and peninsulas which comprise most of the Texas coast. The barrier island complex has grown from 1 to 6 miles wide since its formation about 3,500 years ago (Weise and White 1980). Padre Island is the longest barrier island in the United States and extends about 113 miles from the area of Packery Channel to Brazos Santiago Pass, a natural pass (now stabilized) to the southwest. Padre Island is now bisected by Mansfield Pass (Hansen 1960), an artificial inlet dredged to service Port Mansfield. Mustang Island extends about 20 miles from the area of Packery Channel to Aransas Pass, a natural pass (now stabilized) to the northeast. As summarized by Weise and White (1980), the exact origin of these barrier islands is not known, but it is believed that they owe formation to: (1) submergence of offshore bars initially created by sediments brought from rivers during low sea-level stands; these bars would then "roll over" themselves and approach the mainland with a gradual rise in sea level; (2) spit growth from longshore drift; and (3) drowning, with rise in sea level in the area landward of the mainland beach ridge.

North Padre and Mustang Islands are high-profile barriers primarily consisting of thick deposits of fine-grained, predominantly quartz sand (fine-grained sand is considered to have a diameter in the range of 0.125 to 0.250 mm) (Garner 1967, Mason and Folk 1958). Within the study area, the 1- to 3-mile wide barrier island complex faces the Gulf of Mexico with a broad, gently sloping foreshore and backbeach zone, and three or four longshore bars in the nearshore to about 12-ft depth referenced to mean sea level (MSL). The beaches range in width from 200 to 300 ft (Morton and Pieper 1977) and much of the berm near the project site is regularly cleaned and maintained for vehicular access by beach scraping techniques. Landward of the berm, a nearly continuous line of foredunes exists except where the dunes have been breached by hurricane passes or interrupted by beach access roads and north Padre Island's 12-ft high,

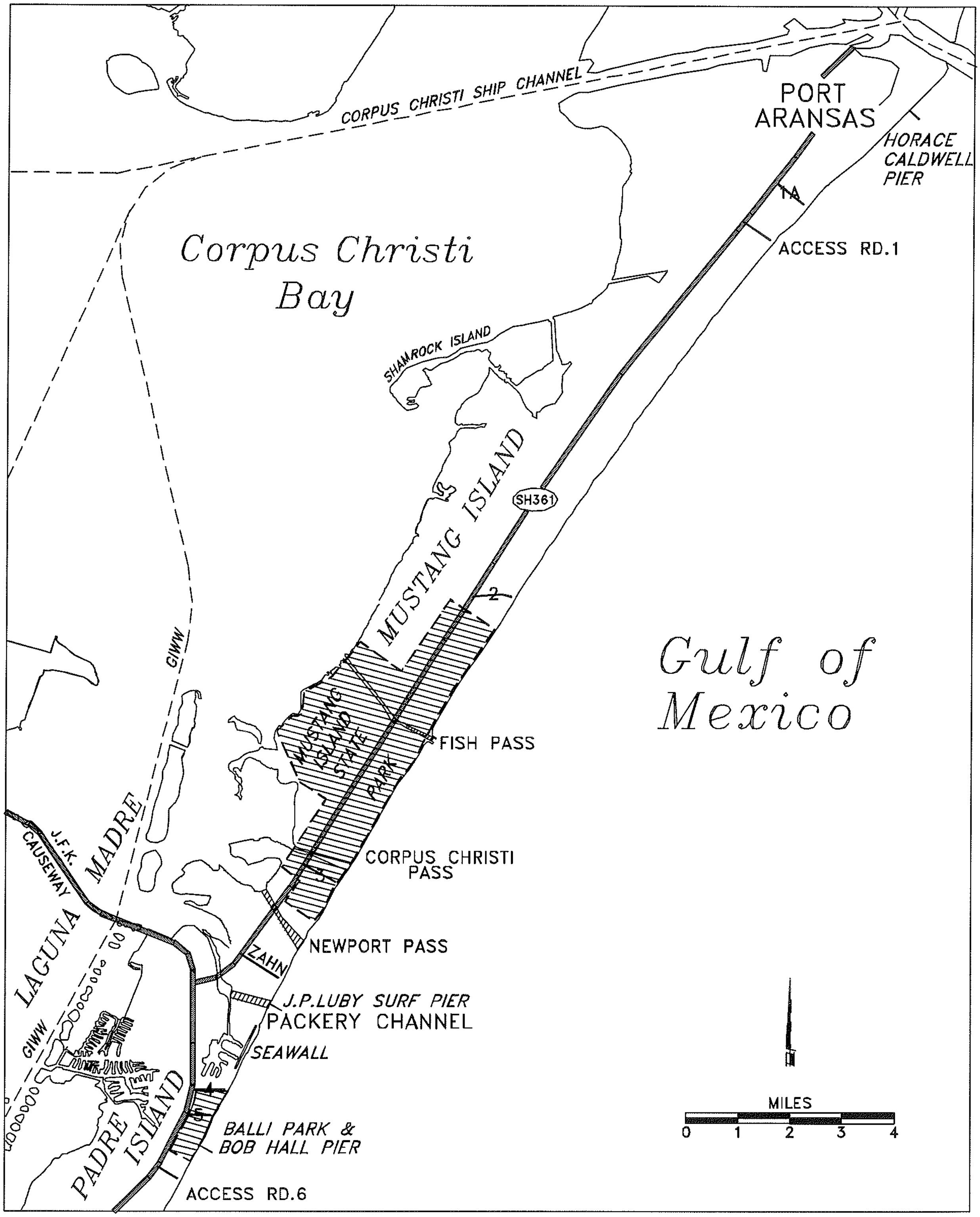

Figure 3. Project area location at the junction of north Padre and Mustang Islands.

4,200-ft long concrete seawall. The foredunes have an average height of about 20 ft (Morton and Pieper 1977), but may reach heights in excess of 30 ft (Davis and Fox 1975), and are backed mostly by cross-island dune fields, vegetated barrier flats, and wind tidal flats (Hayes 1965).

During field work conducted during this study in April, 1996, sediment samples were taken at: (1) the flank of the dune or base of the seawall, whichever was present, (2) mid-berm position, (3) at the position of the mean shoreline at the time of the sampling, (4) at 3-ft depth (in the surf zone), (5) at 12-ft depth, and (6) at 24-ft depth. Results of grain-size analysis of the samples are given in Appendix C.

Particle size of the collected sediment samples was measured by a Malvern Instruments Corporation, Ltd., instrument which operates by laser-light diffraction principles to cover a size range from 0.1 to 600 μm. The light-diffraction method accounts for all grain sizes from clay to medium sand in one measurement procedure. The median grain size of the sediment along the study beach was in the range of 0.17 and 0.20 mm, similar to values previously reported in Mason and Folk (1958), Gage (1970), Garner (1967), and others. There is a trend for grain size to be finer seaward of the surf zone in 12- to 24-ft water depth. The consistent distribution of fine-grained sediment along beaches in the study area indicates that beach-quality sand moving with the longshore current (directed predominantly to the southwest) would fall into and be impounded within a channel at the proposed study site. Such material would be suitable for placement on the beaches adjacent to and fronting the north Padre Island seawall.

Shoreline Change

At present, long-term shoreline change along north Padre Island and Mustang Island is primarily controlled by sediment supply, relative sea-level rise, and hurricanes (Morton and Pieper 1977). Sediment supply to Mustang and north Padre Islands is provided mainly by longshore drift, although a significant quantity of this sediment is lost by landward wind-blown sand transport and hurricane washovers, as well as by impoundment at the jetties and ship channel at Aransas Pass. The long jetties (over 1.7 miles) and deep ship channel (maintained to a depth of 47 ft) at Aransas Pass effectively intercept sand moving alongshore that would have otherwise bypassed the inlet periodically in a geologic time frame (hundreds of years) to nourish the beaches along Mustang and north Padre Islands. This sediment sink contributes to long-term rates of Gulf shoreline recession in the study area of about 5 ft/yr (Morton and Pieper 1977 and Paine and Morton 1989) (Figure 4), rates which were verified for the study area in the present work using November, 1968, and January, 1996, digitized aerial photographs (Figure 5). An unknown percentage of the sand impounded at Aransas Pass is returned to the littoral system after routine maintenance dredging by the USACE, for which beach-quality dredged material is placed in an offshore disposal site located southwest of the channel (USACE 1992).

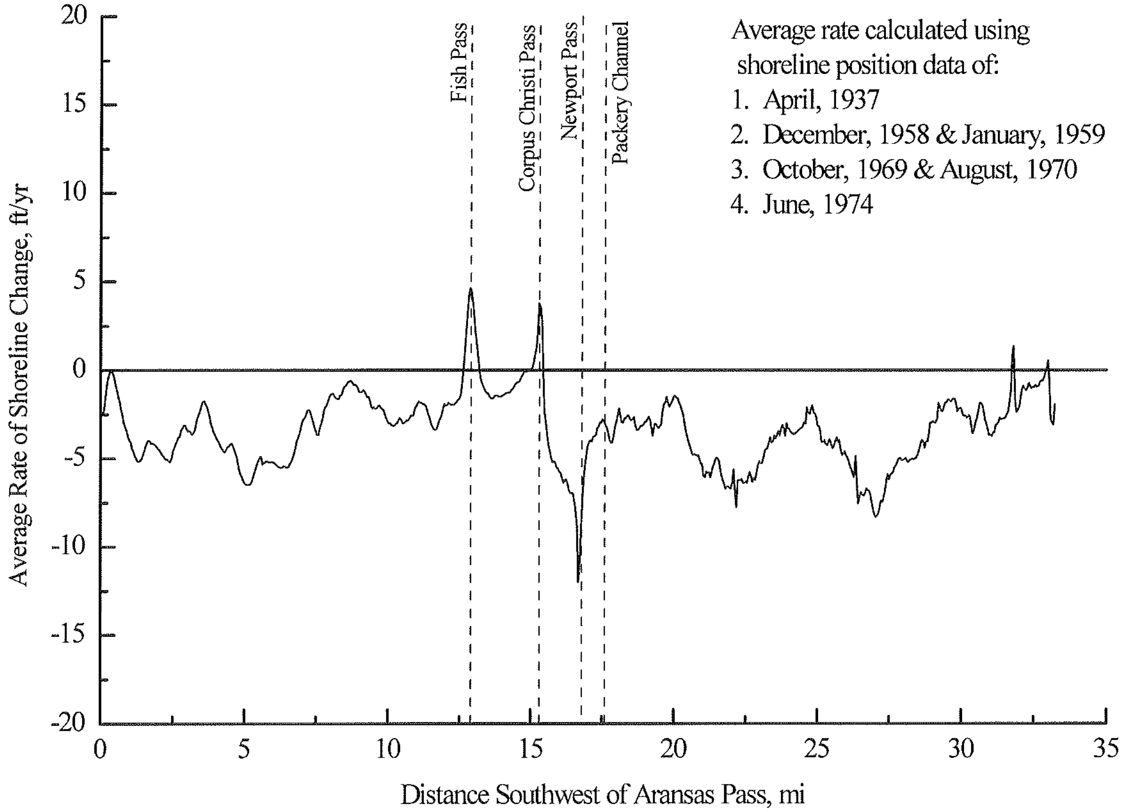

Figure 4. Rate of change of shoreline position, 1937-1974.

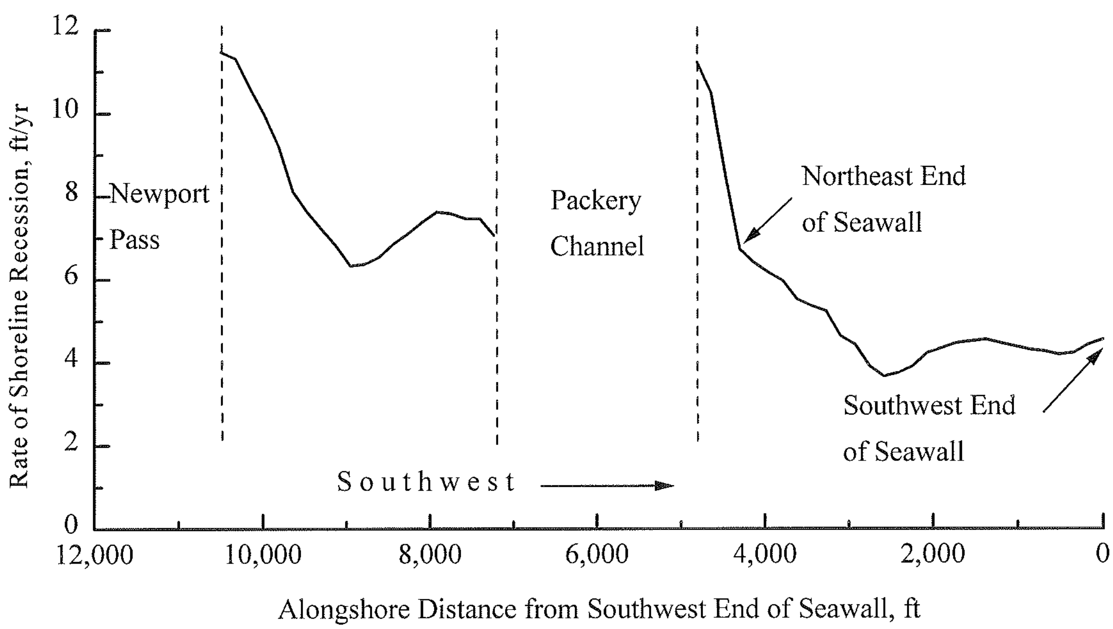

Figure 5. Rate of shoreline recession near Packery Channel calculated using November, 1968, and January, 1996, aerial photography.

Mustang Island Fish Pass

The Mustang Island Fish Pass was a 2-mile long channel dredged and opened in August, 1972, by the Texas Parks and Wildlife Department to increase water exchange and fish migration between Corpus Christi Bay and the Gulf of Mexico (Figure 3). Although the pass was conceived as an environmental benefit, initial consideration of small-craft navigation requirements led to the following design channel dimensions: a channel length of 10,000 ft measured from the bay entrance to the landward ends of the jetties; a bottom width of 60 ft, a top width of 120 ft, and a depth of 8 ft (MSL); a jetty spacing of 400 ft and length (measured from the natural shoreline) into the Gulf of 870 ft; and an entrance-channel bottom width of 100 ft, top width of 150 ft, and depth of 11 ft (MSL) (Behrens *et al.* 1977). Although the entrance channel depth was 11 ft, the jetties probably did not transverse the outer bar, which has a natural crest depth of about 7 ft, and were considerably shorter than the original recommended design length of 1,400 ft (Turner, Collie & Braden, Inc. 1967a, 1970). The channel shoaled to about 4 ft MSL within 3 months after opening, but remained relatively stable at this depth without maintenance dredging and despite the relatively short jetties; by March, 1985, the pass had closed due primarily to shoaling near the Gulf entrance (see Figure 21).

The Fish Pass can be viewed as a successful project in that it remained open for approximately 12 years (Goldston Engineering, Inc. 1985) without maintenance dredging and despite its short jetties. According to Carangelo[2], eye-witness accounts indicate the pass closed in 1981 and that it remained effectively open for only 9 years. As noted by Behrens (1979) and Behrens and Watson (1977), the rate of closure of the Fish Pass was slow, and it exhibited a capability to flush "short-term sediment loads." Comprehensive studies on the hydraulics and dynamics of the Mustang Island Fish Pass are presented in Behrens and Watson (1973), DeFehr (1973), Watson and Behrens (1976), and Behrens *et al.* (1977). Note that the pass has been referred to in the literature as New Corpus Christi Pass and Corpus Christi Water Exchange Pass. Reviews of the reasons for closure of the pass and an exploration of the feasibility of reopening and maintaining the pass were presented by Goldston Engineering, Inc. (1985a, 1985b) and Shiner, Moseley and Associates, Inc. (1987).

As documented by Behrens *et al.* (1977) and Watson and Behrens (1976), the short jetties (which extended approximately 870 ft into the Gulf to a depth of 8 ft MSL) acted as groins, and an efficient natural bar-bypassing system was quickly established to transport sand alongshore and past the inlet entrance. Despite this bypassing system, short-term erosion occurred along the beaches adjacent to the jetties as sand was lost into and through the pass. Although Behrens *et al.* (1977) reported that the erosion occurred downdrift (south) of the system during the first

[2] Personal communication, November 1, 1996, Paul D. Carangelo, Coastal Environmental Planner, Department of Engineering Services, The Port of Corpus Christi Authority.

study year, Watson and Behrens (1976) reported net beach erosion updrift (north) of the system overall for the three-year study period. The following explanation for the updrift erosion is given in Watson and Behrens (1976): *"This formation was apparently due to the tidal discharge asymmetry caused by intracoastal wind tidal circulation and by longshore transport reversals such that net northward (updrift) transport coincided with maximum flood discharges and net southward (downdrift) transport coincided with maximum ebb discharges."* The tidal discharge asymmetry (the dominance of flood over ebb tides) apparently resulted in the interception of sand by the jetties during periods of northward longshore transport and a flood-dominated tide (Summer), and uninterrupted transport of sand to the beach south of the jetties during periods of southward longshore transport and an ebb-dominated tide (Winter).

Shiner, Moseley and Associates, Inc. (1987) noted additional factors which may have contributed to the closing of the Fish Pass, including (1) placement of large quantities of dredged material upwind of the channel where it could readily blow back into the channel; (2) lack of maintenance dredging; and (3) failure to install adequate channel shoreline protection to prevent channel meandering and flanking of the jetties at their landward ends.

Another factor contributing to the infilling of the Fish Pass was the increasing width of the channel through time, which provided sediment to create shoals and decreased the hydraulic efficiency. The erosion of sidebanks indicates that strong currents frequently passed through the original channel. This process led to the bulkheading of a portion of the bank along a bend in the channel in July, 1974 (Watson and Behrens 1976).

Yarborough Pass

In addition to the Mustang Island Fish Pass, other artificial passes have been cut in the vicinity of the proposed project site to moderate salinity in the Laguna Madre or to provide a means for marine organisms to enter and exit the Laguna Madre and Corpus Christi Bay (Collier and Hedgpeth 1950). Of these, Yarborough Pass (formerly called Murdoch's Landing Pass) lies about 30 miles south of the project site and between the proposed Packery Channel and Mansfield Pass, in the vicinity of "Little Shell Beach" and a convergence zone (as outlined in Weise and White 1980) of longshore sediment transport. Morton and Pieper (1977) record five attempts at opening Yarborough Pass, and two aerial photographs of the pass (open and closed) appear on the cover of their report. These openings, apparently all sponsored by the former Game, Fish, and Oyster Commission of Texas (now incorporated in the Texas Parks and Wildlife Department) as temporary measures to reduce salinity in the Laguna Madre, were not engineered in considering required depth, width, placement of jetties, maintenance dredging, and other actions that can be taken to promote inlet stability. Instead, they were simple cuts made with limited resources and equipment through a wide barrier island and connecting the Gulf to a hypersaline lagoon of very small tidal prism. In this context, the rapid closing of Yarborough Pass cannot be compared to an engineered project such as the proposed Packery Channel.

The material in this paragraph illustrates the level of effort involved in opening Yarborough Pass in February, 1952, and was kindly provided by Mr. James Goldston[3], of Goldston Engineering, Inc., who conducted the dredging. *"In the early 1950s, Sun Oil had dredged eastward about 600 ft from the GIWW; this channel, which was about 8 or 9 ft deep and 70 to 80 ft wide, still exists. The Game, Fish, and Oyster Commission provided $40,000 to complete the channel to the Gulf as a means of freshening the water in the lagoon. We used an 8-inch dredge, which had insufficient capacity for such a job, and reached to the southwest side of the dunes, pumping only 1,500 to 2,000 cu yd per day. Because there were no levees, water and sand would find its way back into the channel, and we weren't making any progress. So, we shut down the dredge and brought in a drag line. We took the drag line to the northeast side of the dune line to about elevation of 2 ft above mean tide level. Then we brought a demolition team in from Houston and blew open the channel. For the demolition, we waited until there was high tide in the lagoon and low water in the Gulf so that the tidal current would carry material out. When the channel was blown open, the current was so swift that a sounding survey boat back in the channel was swept into the Gulf. I believe the initial channel entrance opening was 6 ft deep and 70 ft wide. For all this work we were on site 2 to 3 months. While Yarborough Pass was open, we found that the water in the Lagoon near the channel was freshened, and you could catch a yellow-mouthed fish not seen in the Laguna Madre. The channel stayed open about 5 months and first closed on the lagoon side. Once water stopped flowing, littoral drift closed the mouth. Six months later you could drive down the Gulf beach and not know that a channel had been there."*

Mansfield Pass

Based on channel dimensions and jetty size, the Port Mansfield Entrance Channel (located about 70 miles south of the study area) is considered to be an appropriate example of a maintained Texas coastal inlet for prediction of shoaling at Packery Channel. Originally dredged and jettied in 1957 by the Willacy County Navigation District, the Port Mansfield Entrance Channel closed by 1961 due to subsidence and the high void ratio of the jetties (Kieslich 1977). By 1962, the channel had been re-opened by the USACE with improved jetties extending approximately 1,400 ft into the Gulf (measured from the 1962 shoreline position[4]) and with a 27-ft deep, 100-ft wide (at bottom) entrance channel extending approximately 2,000 ft into the Gulf

[3] Personal communication, September, 1996, Mr. James Goldston, formerly President, Goldston Engineering, Inc.
[4] The shoreline updrift of the channel advanced 750 ft between 1957 and 1962 (Kieslich 1977).

(Kieslich 1977, McLellan[5]). Since 1980, the channel has been maintained by contract pipeline dredging to a depth of 16 ft. The original 27-ft dredging depth was required to accommodate the doors of the government hopper dredge that maintained the channel entrance. The jetties and the periodically maintained ship channel effectively block sediment movement along the shore, although some of the dredged material reaches the downdrift beaches after being placed in an offshore disposal site located north of the channel and on the beach adjacent to the north jetty (USACE 1992), and a fraction of the finer suspended-load sediments naturally bypasses the channel. As discussed in a later section of this chapter (see *Waves and Longshore Sediment Transport*), the net direction of littoral sediment movement in the Port Mansfield area is to the north. This net northward longshore flow of sand has resulted in considerable shoreline offset at the channel as littoral sediment has been impounded against the south jetty and eroded from the beach north of the channel (Figure 6). Limited measurements of the current in the entrance channel at Mansfield Pass (Kieslich 1977, Ward 1981) suggest the ebb current tends to be stronger than the flood current in the Summer, probably due to water transported northward from the Lower Laguna Madre under strong southeast wind (Militello and Kraus 1995). Keislich found the flood current to be stronger in the Winter. In a study of seven jettied inlets along the Texas coast, Morton (1977b) found that an ebb-tidal shoal does not tend to form at Mansfield Pass. Apparent absence of an ebb-tidal shoal probably owes to the microtidal range (weak tidal flow) and absence of fresh-water inflow to the tidal basin.

In Figure 6, a suspended sediment plume is being transported past the south jetty at Port Mansfield by the northward moving current. This plume is expected to consist primarily of fine material (silt and clay) suspended by breaking waves, and not beach sand. Silt and clay do not remain in the surf zone, but move offshore into less energetic water. The following demonstrates that most sand moving in suspension around the jetties at Port Mansfield will fall into the channel, by reference to Figure 7.

Figure 7 is a schematic of the cross-section of the water area and bottom as facing the Port Mansfield jetties from the Gulf of Mexico. The channel is 300 ft wide at the top and 16 ft deep, and it is located approximately in the center of the jetties that have an approximate 1,000-ft spacing between them. A representative depth at the tips of the jetties is 10 ft.

Let us assume that a sand grain of 0.17-mm diameter, a typical grain size for the beaches along Padre Island, is suspended by breaking waves to the top of the water column (10 ft above the ambient bottom at the tips of the jetties). A sand grain of that size will have a fall speed w of about 2 cm/sec or 3/4 inch/sec. A representative longshore current velocity V moving the sand particle is 0.3 m/sec or 1 ft/sec. This is a rather strong current on average, but gives a convenient

[5] Personal communication, June, 1996, T. Neil McLellan, Hydraulic Engineer, USACE, Galveston District.

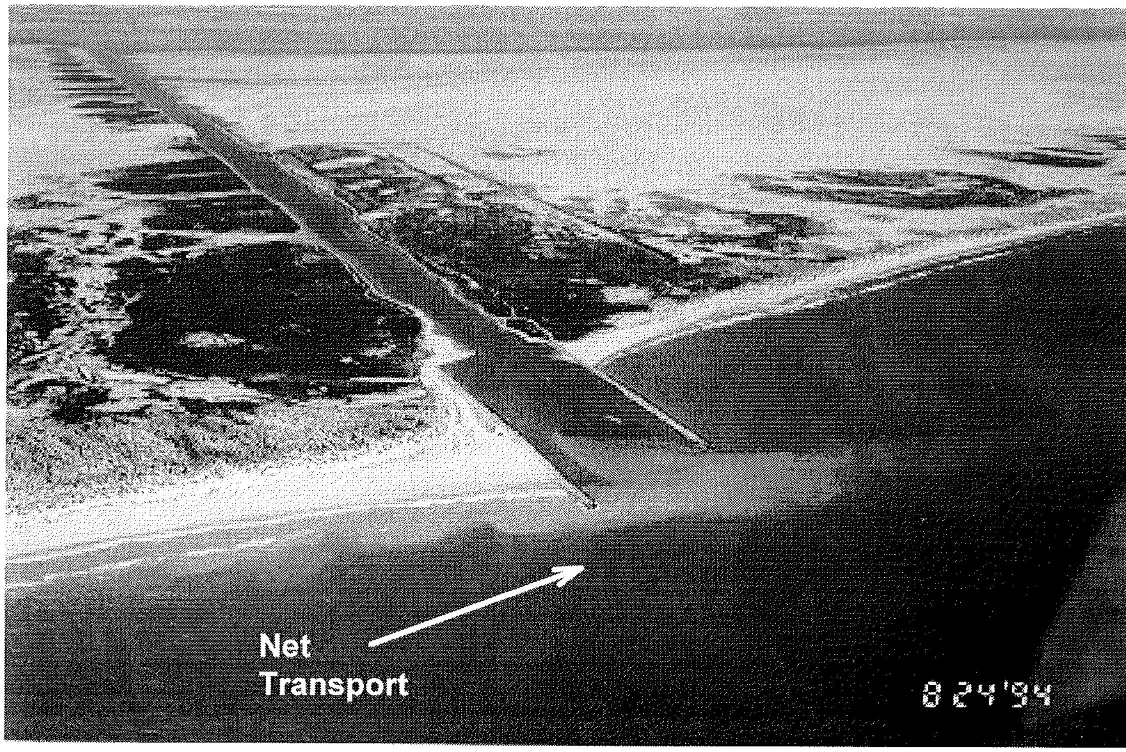

Figure 6. Jetties at Mansfield Entrance Channel, August 8, 1994, looking northwest.

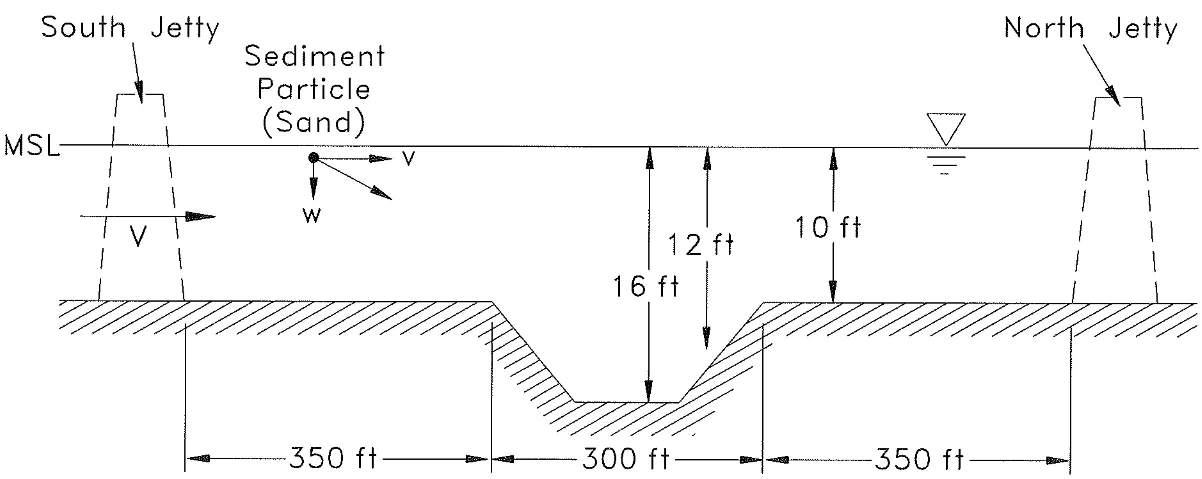

Figure 7. Schematic of the cross-section of the Port Mansfield jetties as facing from the Gulf.

number. Assuming also that the sand grain has to fall 12 ft through the water before being trapped in the channel, the time it will take the sand particle at the water surface to fall and become trapped is about 200 sec. In 200 sec, the sand particle will have been transported alongshore only 200 ft by the current. Therefore, for this somewhat energetic typical condition, the sand particle will fall to the bottom prior to reaching the channel and then be transported into the channel by moving along or near to the bed.

For a storm condition, a corresponding strong longshore current velocity would be 5 ft/sec. In this situation, a sand particle at the top of the water column is expected to move on the order of 1,000 ft and would pass over the channel to reach the other side of the entrance. We conclude that some sand will bypass the Mansfield jetties if there is a wide surf zone and a strong longshore current. This conclusion indicates that the amount of material dredged from Mansfield Pass, as discussed next, might be an underestimate of the gross rate of longshore sand transport. However, it should be noted that the bypassed amount is expected to be small because the above calculations were done to obtain the maximum possible transport distance for grains at the top of the water column. Also, sand dredged from a channel contains contributions that are not related to longshore transport, but to, for example, wind-blown sand, and to-and-fro sand motion that has a net transport of zero.

Both Kieslich (1977) and the USACE, Galveston District, are sources for records of material volume dredged from the Entrance Channel in the individual operations conducted between 1962 and 1973. Because discrepancies exist between the data sources, Kieslich[6] has indicated that there may be errors in the records published in his 1977 report. Therefore, the USACE records are considered to be more accurate and were analyzed in the present study. Table 1 is a summary of the maintenance dredging that has occurred at the Port Mansfield Entrance Channel since 1962. This summary includes maintenance dredging in sections of the channel seaward of Station 3+000 (Station 3+000 is approximately 3,000 ft landward of the 1962 shoreline position). Where a total dredged material volume was recorded by the USACE for a single channel section that was subjected to both new-work and maintenance dredging, the percentage of channel length that was subjected to maintenance work (for that particular channel section) was multiplied by the total dredged material volume (for that particular channel section) to estimate the maintenance dredging volume.

Average annual dredging rates listed in Table 1 were calculated by dividing the total volume of material dredged during the period of interest by the total time interval of that period. Best-fit curves were calculated for values of cumulative volume dredged as a function of time for each of three periods (1962 to 1979, 1979 to 1994, and 1962 to 1994) to determine dredging volume

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[6] Personal communication, June, 1996, James M. Kieslich, Hydraulic Engineer, USACE, Galveston District.

variance from the trend, also listed in Table 1. The standard deviation of the variances for each of the three periods was then calculated to estimate a representative value of variance in maintenance dredging volume. The values of standard deviation listed in Table 1 include contributions to channel infilling by tropical storms and are not representative of typical (non-storm) conditions. In order to estimate variance in entrance channel maintenance dredging volume under typical conditions, unusually high dredging volumes following passage of tropical storms should not be included in calculation of the standard deviation for the period of interest. Following the occurrence of most tropical storms, as listed in Appendix G, the subsequent two dredging operations removed unusually large volumes of sediment (e.g., after Hurricane Allen, August, 1980, and Tropical Storm Jeanne, November, 1980).

Based on data available at the Galveston District, an average of approximately 175,000 cu yd/yr of material were deposited in the Port Mansfield Entrance Channel from May, 1962, to April, 1994. The rate calculated by Kieslich (1977) for individual dredging operations between 1962 and 1973 was 390,000 cu yd/yr, now considered erroneous. Because the entrance channel was dredged to 27-ft depth prior to December, 1980, and was trapping additional sand not associated with longshore transport, the volume of sand infilling during that period is not representative of the volume of sand that would be trapped in a shallow-draft channel along the Texas coast. In inspecting Table 1, for transfer of estimated channel maintenance dredging requirements, the average dredging rate and standard deviation for the period 1979-1994 correspond most closely to anticipated conditions at the proposed channel, because during this period the channel was maintained to 16-ft depth. However, to be conservative, the average dredging rate representative of the entire period of record (1962-1994) was selected as the predicted dredging rate at Packery Channel. The estimate is also considered conservative in that the dredging record in Table 1 includes channel infilling by wind-blown sand, which is expected to be prevented at the proposed channel.

By including the large volumes dredged following tropical storm occurrence in calculation of the average volume but eliminating the storm-related volumes in calculation of standard deviation, 175,000 ± 50,000 cu yd are estimated as the upper limit of material that could annually be deposited within the Gulf entrance of Packery Channel under typical conditions. The dredging volume variance (50,000 cu yd) is based on the dredging for typical conditions that occurred after 1979 and is less than the standard deviations listed in Table 1. Additional dredging greater than the estimated typical average annual volume is expected after tropical storm impact in the vicinity of the project site.

Table 1. Maintenance dredging records for the Port Mansfield Entrance Channel.		
Completion Date	**Volume Dredged, cu yd**	**Deviation from Best-Fit Curve, cu yd**
Hopper Dredge: 1962-1979 (Channel maintained at 27-ft depth)		
Nov-63	192,256	84,474
Jan-65	212,971	184,407
Feb-67	42,062	-38,352
Jul-68	361,461	97,365
Jul-70	35,649	-250,102
Jun-71	99,885	-351,036
Aug-71	294,159	-95,137
Jul-72	416,569	102,019
Jul-73	314,900	159,949
Sep-74	311,277	146,905
Sep-74	81,216	228,121
Aug-76	292,433	-66,411
Oct-77	490,800	33,560
Sep-78	226,296	-65,353
Aug-79	354,351	-50,901
Avg, cu yd/yr:	**216,000**	----
Std Dev:	----	**162,000**
Pipeline Dredge: 1980-1994 (Channel maintained at 16-ft depth)		
Dec-80	302,181	155,738
Jul-83	372,765	245,821
May-86	104,196	39,426
Jun-88	132,937	-56,304
Dec-88	169,585	58,365
Mar-91	139,851	-47,856
Nov-91	98,748	-22,629
Apr-94	242,813	-44,494
Avg[7], cu yd/yr:	**130,000**	----
Std Dev:	----	**110,000**
Total Dredging Record: 1962-1994		
Avg[7], cu yd/yr:	**175,000**	----
Std Dev:	----	**124,000**

[7] Added 300,000 cu yd of sediment to account for entrance channel infilling along an estimated 4,000-ft section of channel in going from maintained channel depth of 27 ft to 16 ft in December, 1980.

Beach Profile

Quantitative information about the shape of the beach profile (1) serves as a direct indicator of local wave conditions, (2) can be combined with historical shoreline position data to estimate beach volume change, and (3) gives an indication of the existing level of storm protection. Knowledge of the existing beach profile shape is also required to develop coastal engineering designs (such as jetties and beach fills) that must include project dimensions and sand volume. Beach profile data are most useful if the profile is measured from landward of the highest elevation at the foredune to beyond the seaward limiting depth of active sediment transport.

At Mustang and north Padre Islands, the seaward limiting depth, d_s, associated with the beach profile is estimated through the use of an analytical method presented by Hallermeier (1983), in which

$$\frac{d_s}{(H_0)_{12}} = 2.3 - 10.9\left[\frac{(H_0)_{12}}{L_0}\right] \tag{1}$$

where $(H_0)_{12}$ is the significant wave height in deeper water exceeded 12 hr per year, and L_0 is the deep-water wavelength calculated with the wave period T associated with $(H_0)_{12}$. Use of USACE Wave Information Study (WIS) Gulf Hindcast Station 5 (Hubertz and Brooks 1989) wave data with Eq. (1) provides a seaward limiting depth of approximately 20 ft. This depth appears to represent an over-estimate, particularly if comparison is made to other estimations of d_s documented for harsher wave climates. For Example, Larson and Kraus (1994) determined the depth of closure to be about 20 ft for a beach in North Carolina facing the Atlantic Ocean; the depth of closure along Texas beaches should be less because the average wave climate is less severe. As noted by Kraus *et al.* (1996), over-prediction of the seaward limiting depth of active sediment transport based on Eq. (1) is perhaps an indication of an over-prediction of the wave climate by the WIS hindcast data. Inspection of a series of beach profile surveys at Mustang Island between 1972 and 1975 (Watson and Behrens 1976, and Behrens *et al.* 1977) reveals that active sediment transport occurs to depths of about 15 ft MSL (see Appendix B). However, the accuracy of the historic data is questionable in terms of measuring quantitative changes along the beach profile (particularly for the data collected farthest offshore), because the survey was limited by the available equipment of the time.

Documented beach profile surveys for the study area include Davis and Fox (1972, 1975), who conducted a series of wading-depth surveys along northeastern Mustang Island, Watson and Behrens (1976) and Behrens *et al.* (1977), who conducted a series of detailed surveys at the Mustang Island Fish Pass by sea sled to depths of approximately 14 to 16 ft MSL, and Morton and Pieper (1977) and Morton (1988), who reported wading-depth surveys done in 1975 and 1987 along north Padre and Mustang Islands. As part of the present study, a beach profile survey was conducted at north Padre and Mustang Islands during the week of April 7, 1996, to supplement the sparse profile data and to determine the present shape of the profile for functional design of the jetties. The profile survey was performed by using traditional surveying techniques on land and a sea sled for the portion of the survey conducted in the water, as described by Grosskopf and Kraus (1994). The Blucher Institute sea sled, shown in Figure E8 (Appendix E), contains a prism halo mounted on the 31-ft high mast which serves as the target of an infrared beam emitted from a total survey station. The sled runners slide along the sea bottom so that measurement of the elevation of the prisms located a known distance from the sea bottom

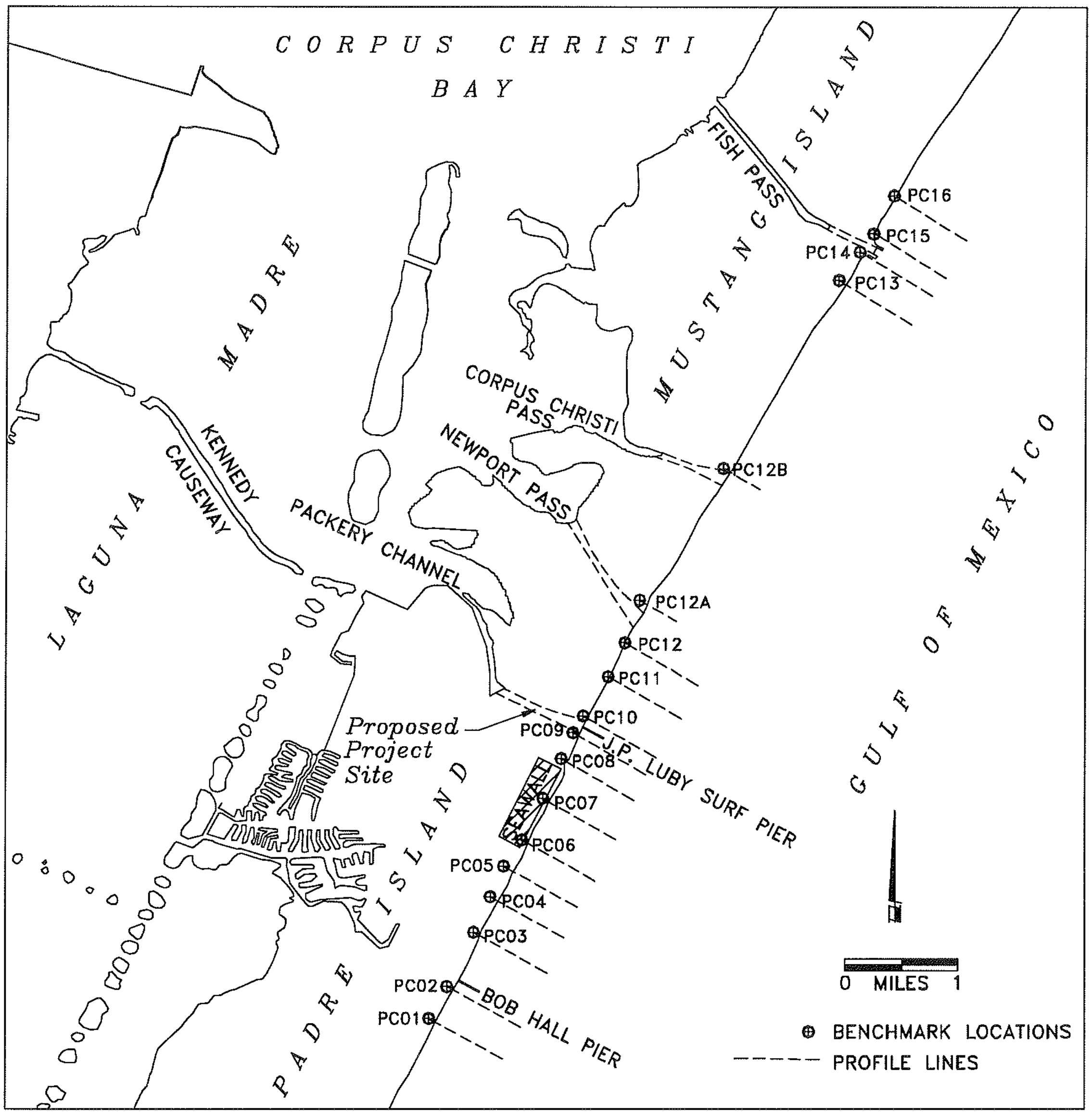

Figure 8. Location of survey benchmarks and profile lines.

can be made. The total survey station fixes the position of the target prism in 3-dimensional space. Accuracy of the vertical measurement is estimated to be on the order of ½ inch, and horizontal accuracy on the order of ½ inch for a stationary sled and 1 to 3 ft for a moving sled, depending on sled speed.

To conduct the beach profile survey, 18 deep-driven survey rods capped by benchmarks were set along approximately 8.2 miles of Gulf-front beach as shown in Figure 8 (see also, Appendix D). The establishment of a linear longshore baseline was prevented by the large survey coverage alongshore and the general curvature of the shoreline. The survey rods were driven to 20- to 60-ft depths and were located typically along the base of the dune line. The benchmarks are expected to survive the impact of a hurricane.

Nine of the survey benchmarks were located southwest of Packery Channel, and the southwestern-most benchmark was emplaced approximately 2.9 miles from Packery Channel along the Nueces County-Kleberg County line. The alongshore benchmark spacing in the southwestern shoreline reach is roughly 2,000 ft, except near Packery Channel where the nominal spacing is 1,000 ft. Two of the benchmarks were driven along the top of the north Padre Island seawall. The northeastern shoreline reach extends approximately 5.3 miles from Packery Channel. Three of the benchmarks are located within one mile north of the proposed channel at a spacing of 2,000 ft. Benchmarks were also driven at Newport Pass and Corpus Christi Pass (1 and 2.4 miles north of Packery Channel, respectively). The remaining four benchmarks are located in the positions of former survey range lines near the Mustang Island Fish Pass (Watson and Behrens 1976 and Behrens *et al.* 1977).

Except for at the Newport Pass and Corpus Christi Pass benchmark locations, the survey lines extended from the top of the dune or seawall to a depth of approximately 25 ft MSL and to a distance of about 3,500 to 4,000 ft seaward of the benchmark (see Appendix B for profile data and Appendix D for benchmark location maps). At Newport Pass and Corpus Christi Pass, the surveys extended only to wading depth. The survey lines were aligned normal to the circumference of a circle of radius 176.4 miles that was fit to the concave trend of the shoreline along north Padre and Mustang Islands. Profile elevation was referenced to MSL based on a National Ocean Service (NOS) primary benchmark at Bob Hall Pier, and the survey lines each contained a nominal 275 distance-elevation pairs (excluding the wading-depth profiles) with greater density closer to shore.

The profile survey data were reduced from raw field format and plotted and cleaned using the Interactive Survey Reduction Program (ISRP), a USACE program for personal computers (Birkemeier and Holme 1992). The overlapping regions of the land and sled surveys were compared, and good agreement was found for all survey lines. The reduced survey set was therefore appropriate for further analysis.

All nearshore (to 25-ft depth) profile lines contained three longshore bars, with four bars present on some lines. Davis and Fox (1972, 1975) described the following three-bar system typical along this portion of the Texas coast (measured from the mean high tide line): (1) an inner bar located approximately 230 to 280 ft offshore at a depth of 3.3 ft; (2) a second bar located approximately 500 to 600 ft offshore at a depth of 4.5 ft; and (3) a third bar located approximately 800 to 900 ft offshore. A plot of the 16 sled-measured profiles aligned to a common shoreline position (MSL) (Figure 9) is in agreement with this description. Note in this figure the 50-to-1 distortion between the scales of vertical and horizontal axes. This figure shows the longshore spatial consistency in the profile shape for stations along open-coast locations of the survey reach. The bars had a slight tendency to approach the shore near the jetties at Mustang Island Fish Pass, which indicates a steeper profile shape. Visual observations at the time of the profile survey were in agreement with the generally steeper nearshore beach face at the Fish Pass. This steepening was not observed for profile survey lines located at or near other coastal structures within the study area (the center and southwest areas of the north Padre Island seawall and 700 ft southwest of Bob Hall Pier). The consistency in profile shape alongshore indicates that the ocean bottom contours are parallel to the shoreline.

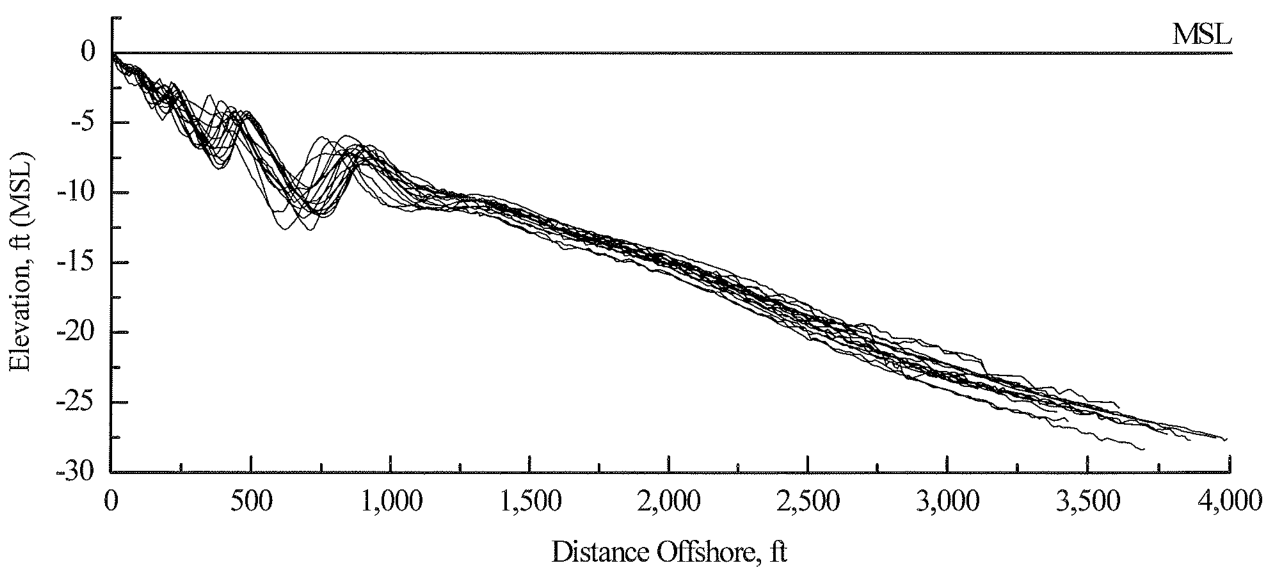

Figure 9. Profiles translated horizontally to a common shoreline.

Several profile survey lines re-occupied the positions of a series of beach profile surveys conducted by Watson and Behrens (1976) and Behrens *et al.* (1977). Profiles PC13, PC14, PC15, and PC16 were surveyed at locations 2,000 ft south, 150 ft south, 150 ft north, and 2,000 ft north, respectively, of the Mustang Island Fish Pass so that a quantitative comparison could be made between the present profile shape and that in 1972, 1973, and 1974 (Appendix B).

Although (as previously mentioned) the accuracy of the historic data was limited by the available survey equipment of the time, the comparison provides several useful insights about the dynamics of the local beach profile. The comparison supports the concept that the bar system is continuously moving with changes in the wave climate within a limited cross-shore range of the profile as controlled by the seasonal wave conditions. At the Fish Pass, this range is approximately 500 ft to 1,200 ft offshore (measured from the MSL shoreline position) for the seaward-most bar and is in the same general location along the historic and present profile shape. Beyond 1,200 ft offshore, any significant vertical variance (e.g., Profile 2000N in Appendix B) in the profile shapes measured in the 1970's is attributed to the limited accuracy of the survey method. The general slope of the profile is constant over time, which supports the use of empirically-derived equations to approximate the average profile shape for modeling purposes.

To calculate an average profile for the beach, the 1996 profiles were translated across-shore to coincide at the shoreline (0 elevation MSL). The calculated average profile and the calculated equilibrium best-fit profile are shown in Figure 10. The equilibrium profile is an idealized profile shape often used in beach profile analysis, and its form is given as

$$h = Ax^{2/3} \tag{2}$$

where h is the still-water depth (referenced here to MSL), A is an empirical shape parameter that depends on the grain size of the beach (Moore 1982, Dean 1991), and x is the horizontal distance from the shoreline to the location of the depth h.

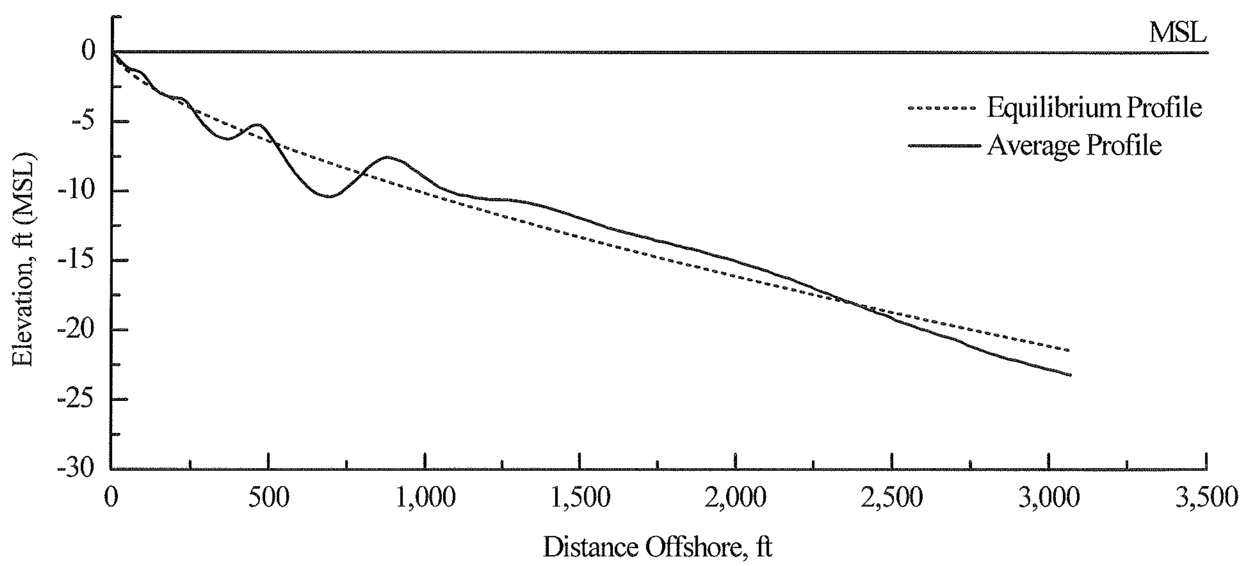

Figure 10. Average profile (from Figure 9) and best-fit equilibrium profile.

The average profile has a dominant bar (called the outer bar or storm bar) located about 900 ft offshore in depth of about 8 ft MSL. Another bar is located about 500 ft offshore in a depth of about 5.2 ft MSL. The best-fit equilibrium profile corresponded to a median grain size of 0.18 mm, which agrees well with values typically in the range of 0.17 to 0.20 mm found from grain-size analysis of sediment samples taken at the time of the profile survey between the foot of the dune and the 3-ft depth contour (Appendix C). Grain sizes of sediment samples collected at the 12-ft and 24-ft depth contours were usually in the range of 0.12 to 0.14 mm. Although the best-fit equilibrium profile may not appear to represent well the region of bars, the discrepancy is only apparent and is an artifact of the approximate 50-times vertical exaggeration of the plot. Overall, the equilibrium profile has a good correspondence to the trend in the average profile shape and is used to represent the profile in the shoreline change model introduced in Chapter 3.

Longshore bars form at a location on the beach profile that is related to the predominant wave breakpoint. This location (water depth) can provide an empirical estimate of the predominant wave height at the site. The bars defined by the average profile are not useful for this purpose because they are smoothed and a mixture of several bars. Instead, the water depth, h_c, over the individual bar crests was determined and averaged. Eight of the survey profile lines were characterized by a four-bar system, with the landward-most bar residing at approximately 1.2 ft MSL depth. This landward-most bar was ambiguously identified on the remaining ten profiles as a swash-zone bar. Three distinct bars were identified on all 16 of the nearshore profile lines. The outer-most bar is formed by the waves accompanying the larger storms, and the middle bar is probably formed by the waves of smaller storms. The one-to-two inner bar system (depending on location) is expected to be associated with the day-to-day common waves. The analysis of h_c gave average values of depth over the outer, middle, and inner (or dual-inner, depending on location) bar crests of 6.9 ft (for the more severe storms), 4.2 ft (for ordinary storms), 2.7 ft (for higher typical waves), and 1.2 ft MSL (for smaller typical waves), respectively.

Larson and Kraus (1989) obtained an empirical expression relating the equilibrium water depth over the crest of bar, h_c, to the breaking wave height H_b as

$$H_b = \frac{h_c}{0.66} \tag{3}$$

after analysis of wave and bar morphology data from large-scale laboratory wave tanks. In the tank experiments, the bars were in near equilibrium with the waves that created them, and they approached their maximum possible volume and height, and minimum depth over the crest. In nature, it is rare for waves of the same characteristics to act sufficiently long to create an equilibrium bar. Usually, the bar will be smaller and the depth under its crest greater than that contained in Eq. (3) which pertains to equilibrium conditions. Therefore, applied to field measurements of the depth over a bar crest, Eq. (3) will tend to overestimate the breaking wave height inferred from morphology.

Based on Eq. (3), a representative significant depth-limited breaking wave height for severe storms in the area is estimated to be 10.5 ft, the representative wave height for ordinary storms is 6.4 ft, and the upper- and lower-limits of the typical wave height are 4.1 ft and 1.8 ft, respectively. These morphologically derived estimates of extreme wave heights can be compared to the results of a wave hindcast described in the *Waves and Longshore Sediment Transport* section of this chapter. Note that these associations of wave height with bar-crest depth should not be confused with the wave heights required for wave breaking over the respective bars. For example, although a wave height of approximately 10.5 ft formed the outer-most bar, individual waves with heights of only 4 to 5 ft will break on that bar.

Significant wave height is defined as the average of the highest one-third of the waves in a record (such as collected during 20 minutes of wave observation). Many of the individual waves in the record will be larger than the significant wave, which is an average. Similarly, even for some observations that have a relatively small significant wave height, there will be higher waves than the significant wave that created, say, the outer-most bar. Therefore, because random waves in the field are comprised of individual waves of different height, period, and direction, it is possible to have occasional wave breaking on the outer-most bar for waves that may, say, be forming the middle bar according to their significant wave height.

Finally, it is mentioned that Eq. (3) should not be confused with the standard criterion for depth-limited wave breaking, which is $H_b = \gamma\, h_b$, where h_b is the depth at breaking. The breaker index γ has a representative value $\gamma = 0.78$, although the index is known to depend on local bottom slope and wave period (e.g., Kaminsky and Kraus 1994). The breaking depth h_b along a bar is located considerably seaward of the location of the bar crest; that is, incident waves feel the bottom and begin breaking at depth h_b, not at the depth over bar crest h_c. Therefore, along a bar, for the significant wave that creates it, h_b will always be larger than h_c. Combining the two aforementioned equations for breaking wave height, $h_c \cong 0.66 \times 0.78\, h_b = 0.51\, h_b$. Eq. (3) is useful for estimating H_b because the depth over bar crest can be uniquely located on the profile, whereas the depth at incipient breaking cannot.

Wind and Water Level

As noted by Lohse (1952) and Watson and Behrens (1970), winds in the study area are strongly bimodal, consisting of persistent southeasterly winds occurring from March or April through August or September and north-northeasterly winds occurring from September or October through February or March. According to Watson and Behrens (1970) and Watson (1971), the annual resultant vector based on velocity, duration, and direction is perpendicular to the central Texas coast approximately along Mustang Island. Northerly winds associated with frontal passages usually are strong and may be accompanied by precipitation. These northers can cause a reversal in wind direction and as much as a doubling in wind speed in a matter of minutes (Collier and Hedgpeth 1950). Davis and Fox (1975) observed wind speeds going from near zero to greater than 34 mph in less than one hour, and that the passage of cold fronts commonly caused rapid and extreme changes in the longshore wind component.

Wind roses were constructed for the study area based on data provided by the National Oceanic and Atmospheric Administration (NOAA), National Weather Service (NWS) in the open Gulf at Horace Caldwell Pier (Figure 11), and by TCOON in Corpus Christi Bay (Figure 12). Note that friction losses and other boundary-layer distortions have little impact on the wind speeds recorded in the Bay as compared to those recorded in the open Gulf. The predominant wind speed in the open Gulf and in the bay is in the 10-to-18 mph range.

Figure 13 shows the relation between tidal datums obtained from the Bob Hall Pier station (located at the Gulf coast of Mustang Island, Figure 3) as available from NOS based on time series of water-level readings made between July, 1985, and December, 1988. Mean high water (MHW) is defined as the arithmetic mean of the high-water heights observed over a 19-year cycle or an equivalent by comparison to a control station, and mean low water (MLW) is defined as the arithmetic mean of the low-water heights observed over the same cycle. Mean tide level (MTL) is the average of MHW and MLW, and MSL is the average of the hourly determinations of tide level. At Bob Hall Pier, MTL is about 0.01 ft below MSL. Definitions of the other tidal datums shown in Figure 13 may be found in Hicks (1986, 1989).

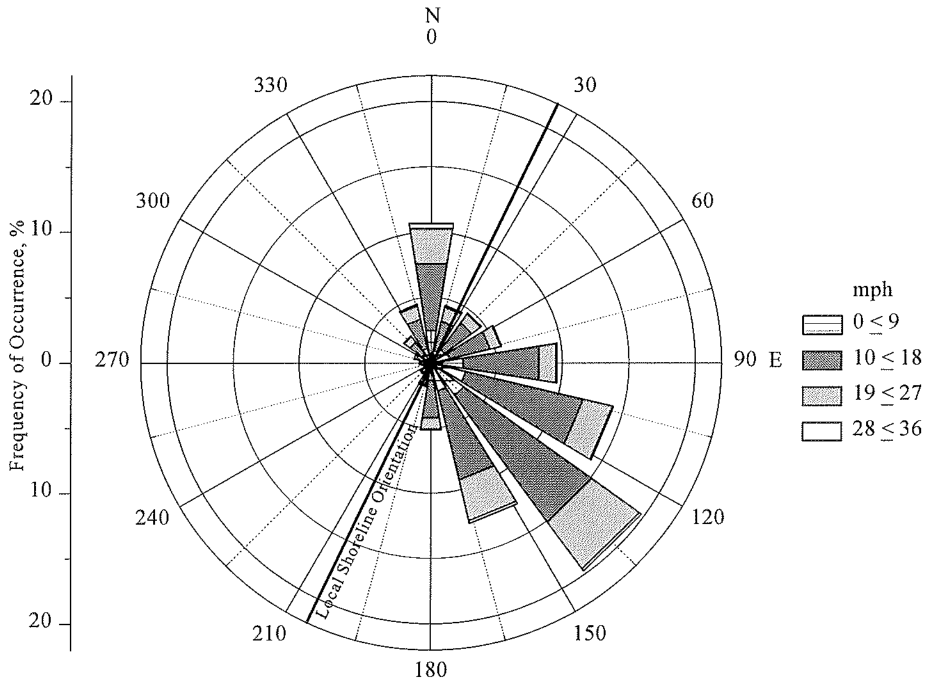

Figure 11. Wind Rose from Horace Caldwell Pier NWS C-MAN station in Gulf of Mexico south of Aransas Pass (at 27.8°N, 97.1° W), 1984-1993.

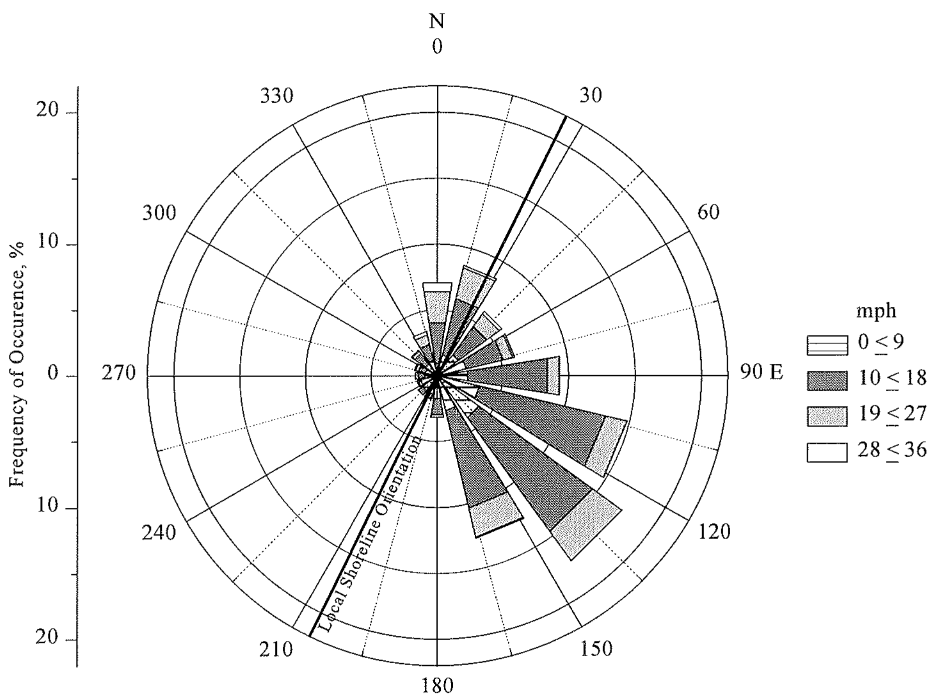

Figure 12. Wind rose from Corpus Christi Naval Air Station TCOON gauge, Corpus Christi Bay, 1995.

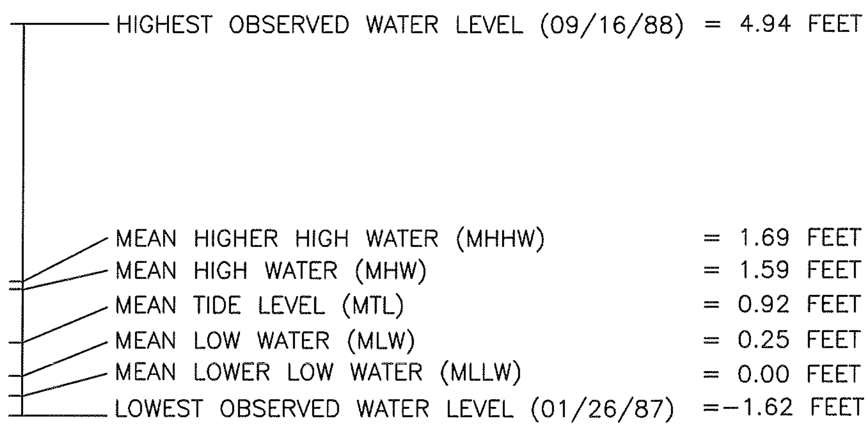

Figure 13. Water-level datums for NOS Bob Hall Pier station 877 5870.

The tide along the Texas coast is mixed, meaning that both semi-diurnal and diurnal signals are present, but, typically, the diurnal component is predominant. Although the (diurnal) tide range is defined as an average of the differences between low and high waters, on the Texas coast the seasonal change in water elevation may be comparable to or greater than the diurnal range, particularly in its shallow bays and estuaries. Figures 14 and 15 plot hourly values of water level recorded at Bob Hall Pier and at Packery Channel. Notice that the water level underwent a broad maximum in spring and autumn, and a broad minimum in summer and winter of 1995. At Bob Hall Pier (which is located in the open Gulf) the tide range is 1.34 ft, whereas at Packery Channel (which is located in the bay) the tide range is 0.36 ft. Because the tide range is defined as the difference between daily high and low waters, on the south Texas coast the tide range does not reflect the much larger range in water elevation that occurs seasonally.

Other motions will be present in a time series of water surface elevation. For example, storms produce surges, and the winter weather fronts that pass through the south Texas coast with an average periodicity on the order of 5 days will also change the water level through the accompanying wind, atmospheric pressure differentials, and movement of water into and out of a particular region.

A relative rise in water level with respect to the land produces an apparent shoreline recession. The long-term trend in the 50-year water level record at the NOS Rockport, Texas, tide station allows estimation of the relative change in water level between water and land near Packery Channel. Lyles *et al.* (1988) reported a mean relative water-level change of 0.16 inches/yr, which gives a relative rise in water level with respect to land of 1.3 ft per century. A similar rise in water level with respect to land on the Gulf side of north Padre and Mustang

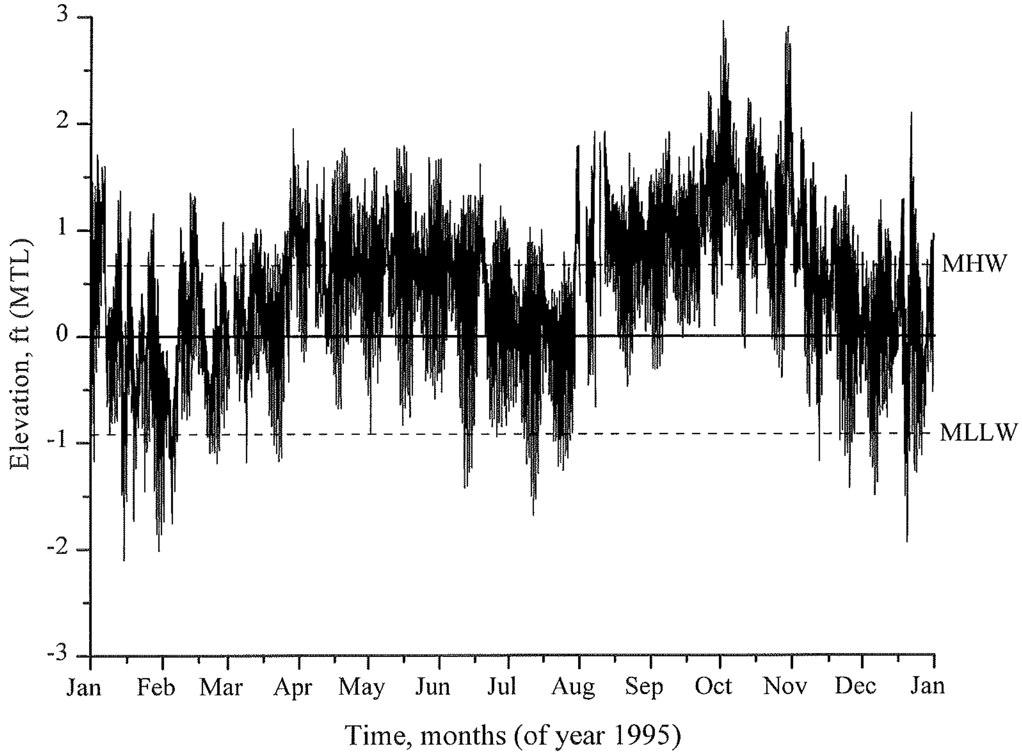

Figure 14. Water level at Bob Hall Pier, Corpus Christi, Texas, in 1995.

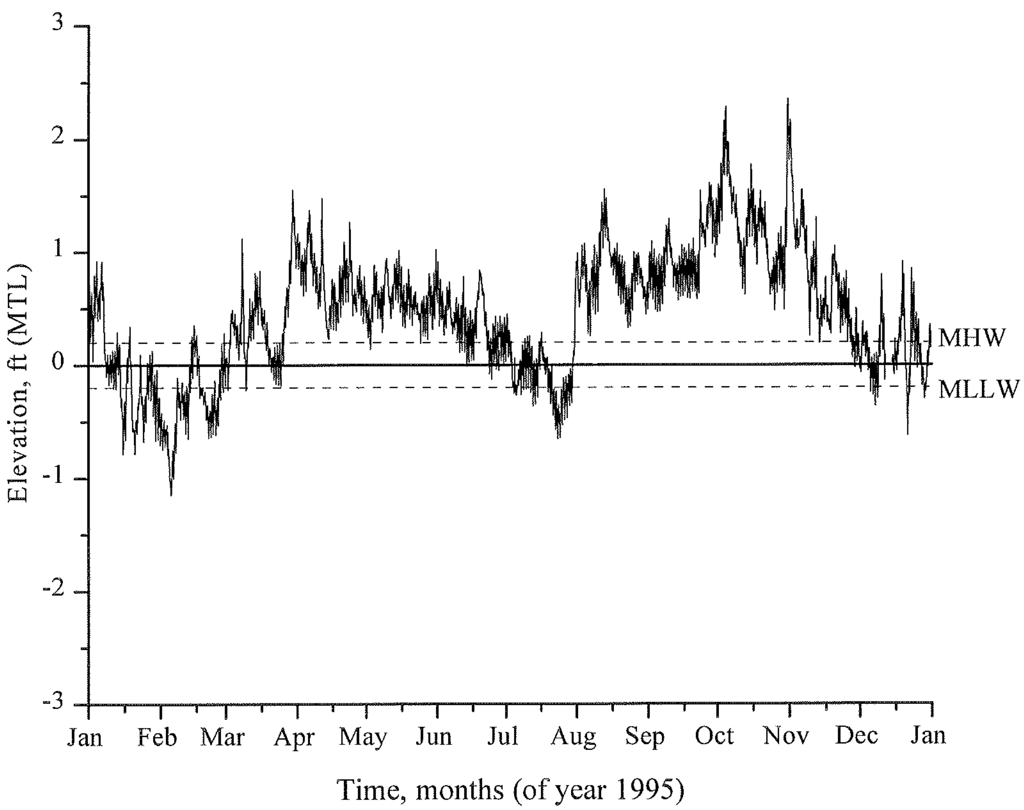

Figure 15. Water level at Packery Channel, Corpus Christi, Texas, in 1995.

Islands is expected. The back beach profile at the study site (from shoreline to toe of the foredune) has a typical slope of approximately 1:100, meaning that the beach drops 1 ft in elevation from the dune per 100 ft moved toward the Gulf. Based on that slope, a 1.3-ft rise of Gulf water with respect to the land would move the shoreline inland some 130 ft, or 1.3 ft/yr on average. This apparent long-term recession continues independently of wave and wind processes (i.e., this is the rate that would occur if only sea-level rise contributed to the recession). The National Research Council (1987) outlines the development and application of methodologies for estimating the expected erosion impact and storm damage associated with relative sea-level rise.

Storms

Hurricanes, which are the most destructive of the natural forces that impact the Texas coast, typically form within the months of June through October. As the storms move landward, large areas are inundated by high surges, attacked by large waves, and subjected to strong winds and tornadoes. According to USACE (1979), hurricanes have struck the Texas coast approximately once every 2.5 years within the last century, and once every 10 years for any specific location. However, a hurricane does not necessarily have to make landfall at a particular location for that location to be impacted by the storm's energy. For example, on August 23, 1996 (during writing of this report), Hurricane Dolly struck the central eastern Mexico coast with wind speeds of 70 mph. This storm radiated large waves to Padre and Mustang Islands, and, combined with heavy rainfall, caused the temporary opening of Packery Channel (Figures 16 and 17). A chronological description of the tropical storms which caused significantly high waves, water-level, or wind along the southeast Texas Coast between 1791 and 1995 is presented in Appendix G. These storms were divided into pre- and post 1900 groups and plotted in Figures 18 and 19. Note that, since 1900, an average of one tropical storm has impacted the study area every two years.

As documented by Price (1956), Corpus Christi Pass and Newport Pass were opened by a tropical storm which struck the Texas coast approximately 30 miles south of Port Mansfield in September, 1933. During Hurricane Beulah, which passed over the south Texas coast near Brownsville in September 1967, Packery Channel and Corpus Christi and Newport Passes were opened together with some 67 other storm-tide inlets and passes across Padre Island (Berryhill 1969). Hurricane Allen, which struck the Texas coast in August, 1980, northeast of Brownsville had a surge that lasted 48 hours and reached a maximum elevation of 8.5 ft on north Padre Island. Allen caused the collapse of several short sections of the north Padre Island seawall (Morton 1988) and resulted in strong flows through the Mustang Island Fish Pass which removed some of the sand deposits from the channel (Figure 21).

Figure 16. Looking along Packery Channel towards the Gulf. Photo taken August 23, 1996, 1:11 p.m., after Hurricane Dolly.

Figure 17. Looking along Packery Channel towards the Bay. Photo taken August 23, 1996, 1:21 p.m., after Hurricane Dolly.

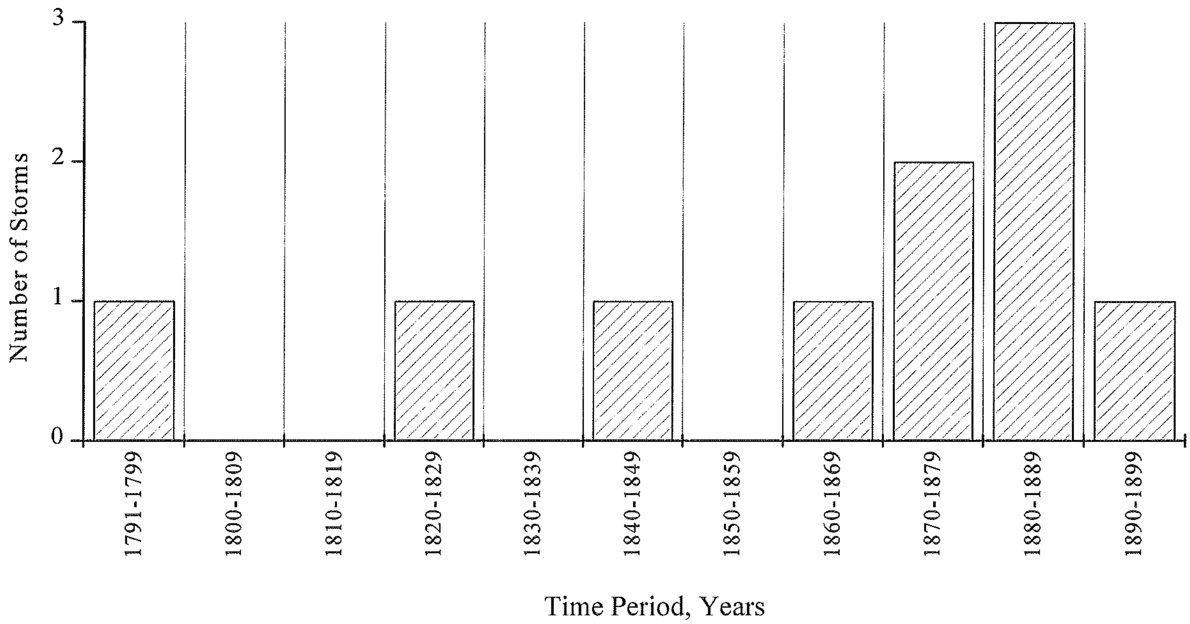

Figure 18. Documented tropical storms impacting the southwest Texas coast prior to 1900.

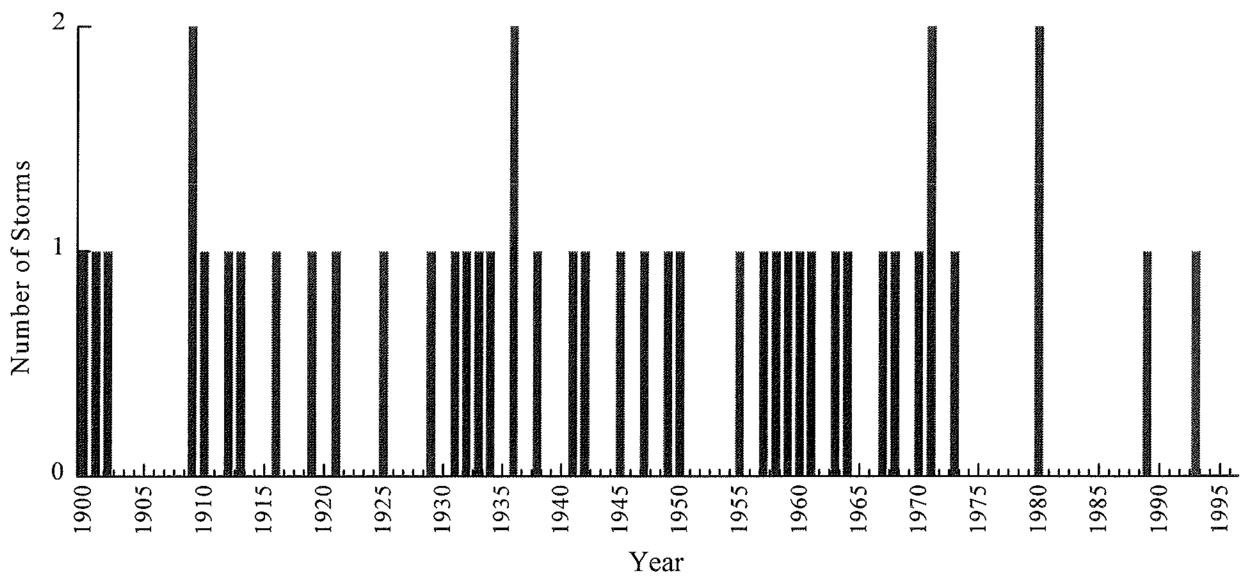

Figure 19. Documented tropical storms impacting the southwest Texas coast, 1900-1995.

Waves and Longshore Sediment Transport

Two longshore sediment transport rates enter prominently in coastal engineering applications, the gross transport rate and the net transport rate (denoted as Q_g and Q_n, respectively). These rates are defined as

$$Q_g = Q_r + Q_l \tag{4}$$

and

$$Q_n = Q_r - Q_l \tag{5}$$

For an observer standing on the beach and facing the Gulf, the gross transport rate is the sum of the right-directed or southward transport rate Q_r and left-directed or northward transport rate Q_l, and the net is the difference of those rates. The proposed channel will trap sand moving from both the left and right that passes the jetties; therefore, the channel potentially traps the gross transport. Shoreline change is governed by the difference in transport, that is, the net transport.

Estimation of longshore sediment transport rates is best determined empirically because of the complex process of waves propagating towards the beach and breaking. Estimates are expected to be reliable within ± 50 percent. The most accurate method of estimation is to transpose a known rate from a nearby segment of shore. Another method of estimating longshore transport rates is to obtain wave measurement or hindcast data for input into predictive formulas.

Although seasonal conditions may cause the net direction of longshore sediment transport in an area to change from year to year, the long-term predominant direction of transport along north Padre and Mustang Islands has been established to be to the south. Evidence supporting the southward net longshore transport includes the observation by Price (1933) that Aransas Pass migrated southward prior to being stabilized by jetties, although studies have documented the migration of inlets in the opposite direction of net longshore drift (Reddering 1983, Aubrey and Speer 1984). Bullard (1942), Van Andel and Poole (1960), and others traced distinctive mineral suites from rivers along the Gulf coast to approximately 27 deg north latitude, suggesting long-term net transport is to the south along Mustang and north Padre Islands and to the north along southern Padre Island. Through a study of geologically effective winds over Gulf waters, Lohse (1952) also concluded that a zone of net longshore transport convergence exists at approximately 27 deg north latitude. Based on wind observations which were used to develop wave hindcasts representative of the eastern and southern ends of and near the central Texas coast, Carothers and Innis (1962) estimated that longshore convergence occurs in the vicinity of central Padre Island. Watson (1971) supported the theory of net convergence near 27 deg north latitude after studying the concentration of shells which had been transported to the area from northern and southern beaches and the wind tidal-flat infill of the Laguna Madre. In a study documenting the historical shoreline positions along the Texas coast, Morton (1977a) noted that although most of the Texas beaches are eroding, central Padre Island has undergone long-term net shoreline advance, supporting the theory of net longshore convergence in that area.

The net rate of transport in the study area is low as indicated by inspection of historical shoreline migration and existing shoreline positions adjacent to local coastal structures. Such inspection was conducted using data of shoreline position adjacent to the Port Aransas Entrance Channel jetties (construction completed by 1916) and Mustang Island Fish Pass jetties (constructed in 1972). Shoreline positions near such structures serve as good indicators of the magnitude and direction of net longshore sediment transport because the structures block the flow of littoral sand; this blockage often results in a significant shoreline offset (shoreline advance updrift of the structure and shoreline recession downdrift of the structure). Because the net longshore sediment transport in the study area is low, there are no distinct long-term shoreline offsets adjacent to local coastal structures. However, as is evident through inspection of published Texas Gulf coast shoreline position data (Morton and Pieper, 1976, 1977, and Paine and Morton, 1989), an advance occurred during the period from 1931 to 1982 at the shoreline updrift (northeast) of the Port Aransas Entrance Channel jetties which exceeded an advance that occurred downdrift (southwest) during the period from 1937 to 1982, supporting the theory that average net transport occurs to the southwest at that location.

Another method for predicting magnitude and direction of longshore sediment transport is through analysis of local wave data. However, nearshore wave gauge measurements are not available for the study area. Other sources of information on the local wave climate are available through observations made 1 mile south of Aransas Pass between July, 1972, and June, 1975, as part of the Coastal Engineering Research Center (CERC) Littoral Environment Observation (LEO) program (Watson and Behrens 1976, and Behrens *et al.* 1977) and through the WIS 20-year numerical simulation hindcast published by the U.S. Army Engineer Waterways Experiment Station, CERC (Hubertz and Brooks 1989). The authors consider these sources of wave data to overestimate the longshore transport rate at the study area (see discussion on pp. 36-37). The 3-year LEO data set, which consists of approximately 880 observations, probably: (1) contains error related to the frequency of data sampling; (2) is weighted towards larger waves, because the data were collected during daylight hours; or (3) contains an unknown systematic error. The WIS data significantly over-predict the nearshore wave height, judging by general observation at the beach and by the analysis of bar crest depth described in a previous section of this report. As noted by Kraus *et al.* (1996), the WIS Gulf Hindcast does not provide accurate data for the western side of the Gulf near the location of the study area (see also Bodge and Kraus 1991).

Instead of relying on previously developed wave data, information on the long- and short-term wave climate was obtained through a hindcast performed by Offshore & Coastal

Technologies, Inc. (OCTI) of Avondale, Pennsylvania, an oceanographic and coastal engineering company that has performed wave hindcasts at sites world wide. Ten years of continuous wave data for the study area were developed using 5-deg pressure fields from 1970 to 1979, a time interval that covers construction of the Mustang Island Fish Pass. Numerical gridding results were developed for a 1-n.m. increment along the Gulf shoreline and significant wave height, peak period, and mean direction were provided about 3,040 ft offshore (at a 32-ft depth). Output was provided at a 3-hr increment at a location approximately 18.4 miles south of Aransas Pass (near Packery Channel) along the Gulf shoreline of north Padre Island.

The OCTI hindcast gave the most frequent wave direction (38%) out of the east-southeast and the next most frequent direction out of the east (30%) (Figure 20); the associated average net direction of longshore transport was southward. The predominant wave direction agrees well in trend with the predominant direction of wind (Figures 11 and 12). The average significant wave height for the hindcast was 1.8 ft, the average peak wave period was 4.6 sec, and the average wave angle was 110 deg measured clockwise from north. The wave hindcast agrees with the morphologically-derived wave height estimate for typical waves of 1.8 to 4.1 ft. The largest significant wave height in the hindcast was 11.8 ft (verifying the morphologically-derived estimate of 10.5 ft) and had a peak wave period of 11 sec.

Table 2 lists longshore sediment transport rates that were calculated using the OCTI hindcast data as a first approximation (prior to shoreline change numerical modeling as discussed in Chapter 4) by means of the SEDTRAN program (Gravens *et al.* 1989). SEDTRAN estimates the potential longshore sand transport rate from the wave energy flux method driven by time series of input wave conditions (significant wave height, period, and direction). The program, which converts the standard engineering quantity of significant wave height to a root-mean square value by the factor involving 1.416, uses a value of 0.77 for the dimensionless proportionality coefficient K (K relates transport rate to the longshore component of wave energy flux) in the CERC formula for the longshore sand transport rate. Note from Table 2 that calculated transport rates (particularly, net rates) for the years 1972 and 1976 are anomalously high as compared to the other calculated rates. The rates for these years were judged by the authors to be unrealistically high for the Gulf of Mexico, and the data associated with those rates were omitted from the wave hindcast data set. Because the yearly rates were computed in a systematic and consistent manner, the unrealistically high rates are likely the result of inaccurate pressure field data available for the wave hindcast. Considering that the study area is near a convergence zone of longshore transport, the acceptable low-range limit of calculated net rates within the data set is zero, and net direction may be either to the north or south. Therefore, no years of low calculated

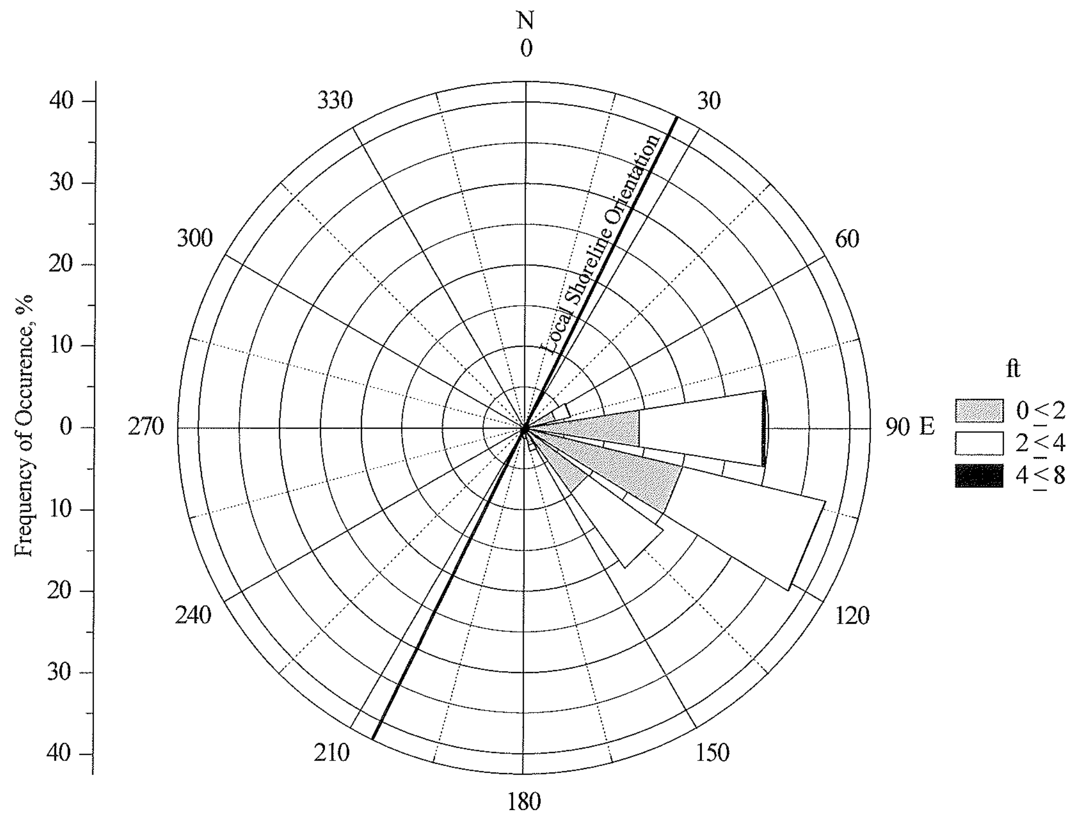

Figure 20. Wave rose constructed from OCTI hindcast data.

rates of longshore transport were omitted from the data set. Alongshore spatial variability in the transport rate is discussed in Chapter 4.

The longshore sediment transport rates calculated using the OCTI hindcast data were compared to rates estimated by Watson and Behrens (1976) and Behrens *et al.* (1977), who based their calculations on the LEO data collected south of Aransas Pass. Because there are several discrepancies between the two reports, the source of the reported average net longshore transport rate of 67,000 cu yd/yr and average gross transport rate of 730,000 cu yd/yr (Watson and Behrens (1976)) is obscure. Kieslich (1977), Mason and Sorenson (1971), Heilman (1995), and others have estimated gross longshore sediment transport rates along the Texas coast which are of similar magnitude (in excess of 500,000 cu yd/yr) to the gross rates reported by Watson and Behrens (1976) and Behrens *et al.* (1977). However, after careful consideration, it was

Table 2. Longshore sediment transport rates calculated based on OCTI hindcast data for location near Packery Channel (rates given in cu yd/yr).

Year	Northward	Southward	Net	Gross
1970	-120,000	220,000	100,000	340,000
1971	-130,000	200,000	70,000	330,000
~~1972~~	~~-30,000~~	~~700,000~~	~~670,000~~	~~730,000~~
1973	-80,000	240,000	160,000	320,000
1974	-90,000	80,000	-10,000	170,000
1975	-40,000	230,000	190,000	270,000
~~1976~~	~~-70,000~~	~~350,000~~	~~280,000~~	~~420,000~~
1977	-130,000	160,000	30,000	290,000
1978	-130,000	160,000	30,000	290,000
1979	-120,000	220,000	100,000	340,000
Avg	-100,000	190,000	80,000	290,000
Std Dev	30,000	50,000	60,000	50,000

concluded in the present study that such rates are unrealistically high for the Texas coast (see also Heilman and Edge 1996), and that representative gross rates are on the order of 150,000 to 250,000 cu yd/yr. This consideration is based on the following:

1. Numerous small, un-maintained inlets have historically existed for time periods in excess of 10 years along the Texas coast, including Rollover Pass (Bales and Holley 1985), Mitchell's Cut (Kraus and Militello 1996), Brown Cedar Cut (Mason and Sorensen 1971), Cedar Bayou (USACE 1992), and the Mustang Island Fish Pass (Watson and Behrens (1976) and Behrens *et al.* (1977)). These inlets, which are all un-jettied (except for the Fish Pass, which had short, ineffectual jetties), would be expected to close rapidly if subjected to annual gross longshore sediment transport rates exceeding 500,000 cu yd.

2. Comparison was made to known rates of gross longshore transport at Atlantic-Ocean and Pacific-Ocean beaches, where greater rates exist than at Gulf-coast beaches because the oceans are much larger than the Gulf and the wave climate more severe. For example, Bodge (1995) determined an average gross rate of about 400,000 cu yd/yr for a beach on the north Florida coast facing the Atlantic Ocean.

3. Recognition was given to the fact that inaccurate data (sometimes including WIS and LEO), inappropriately-applied data, and/or selection of an inappropriate *K* factor for calculating wave energy flux can lead to over-prediction of longshore transport rates. This situation was acknowledged by the coastal engineers who used experience and engineering judgment in calculating longshore transport rates along Galveston Island, Texas, prior to construction of a beach fill in 1994 (Beumel and Beachler 1994). These engineers applied WIS data and then reduced calculated rates approximately by a factor of 3 to estimate an average gross transport rate on the order of 250,000 cu yd/yr[(8)]. The 1994 Galveston beach fill of approximately 1,000,000 cu yd has remained in the general placement area; this would not likely occur if the gross transport rate were in excess of 500,000 cu yd/yr.

In the present study, the following evidence was obtained to provide other independent estimates of the gross longshore sediment transport rate representative of the subject coast:

4. A 31-year record of maintenance dredging volumes (as shown in Table 1) for the nearby Mansfield Pass entrance channel, which contains the impacts of hurricanes and tropical storms, as well as channel infiltration by wind-blown sand, provides a realistic estimate of the upper limit of the gross transport rate as 200,000 cu yd/yr.

5. The OCTI hindcast data used in Chapter 4 as input to a sophisticated longshore transport model (which includes curvature of the shore and other factors) gave a gross transport rate on the order of 180,000 to 250,000 cu yd/yr at Packery Channel.

6. Calculations were made by Turner, Collie & Braden, Inc. (1967a) to predict required annual maintenance dredging at the Mustang Island Fish Pass of 105,000 to 185,000 cu yd/yr.

7. An estimate of gross transport rate at Corpus Christi was made by Lockwood, Andrews and Newnam, Inc. (1959) of 254,000 cu yd/yr using wave hindcast data.

8. An estimate of the gross longshore sand transport rate at Rollover Pass, Bolivar Peninsula, Texas, was made by Bales and Holley (1985) of 150,000 cu yd/yr based on the previous work of USACE (1984), Prather and Sorensen (1972), and others.

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[8] Personal communication, October, 1996, Mr. Norman H. Beumel, Vice President, Coastal Planning & Engineering, Inc., Boca Raton, Florida.

Wind-Blown Sand

Analysis of a time sequence of aerial photographs of the Mustang Island Fish Pass (Figure 21) indicated to the present authors that wind-blown sand was a significant factor in the gradual closure of the pass from its opening in August, 1972, to its complete closure by March, 1985, as pointed out by Duke (1985a) and Shiner, Moseley and Associates, Inc. (1987). Observations of the photographs reveal: (1) Heavy shoaling of the pass occurred adjacent to non-vegetated areas along the dune line instead of at the seaward ends of the jetties, and (2) Sand accumulation adjacent to the jetties was moderate as compared to sand infilling between the jetties. A quantitative field study of wind-blown sand transport was warranted to confirm the above qualitative observations. The field study is described in Appendix F and summarized here.

Known for its consistently strong winds, Corpus Christi is one of the best wind-surfing areas in the world and has become a popular area for professional and amateur competitions, particularly during the warm months from April to August when southeast winds commonly reach speeds of 20 mph by late afternoon. As noted by Price (1956), the local winds also play a significant role in transporting large volumes of beach sand inland.

Because of their high sand trapping efficiencies, water channels are commonly built by agricultural engineers to control dune migration and sand intrusion. In the present study, a series of field sand-trapping simulations (using a sand trap as shown in Figure 22) was conducted because it was clear that, without protection, significant channel siltation at Packery Channel would occur over the course of a year. The physical simulation also demonstrated that the channel siltation will start from the upwind side of the channel and proceed toward the opposite side. A similar behavior was observed at the nearby Mustang Island Fish Pass (Figure 21) where shoals formed from winds out of the southeast in summer and out of the north and northeast in winter.

Based on the results of the field measurements, the following empirical equation was developed as a predictive formula for estimating wind-blown sand transport at Packery Channel:

$$q = 0.0063\ (U_1^3 - U_{1C}^3) \qquad \text{for} \qquad U_1 > U_{1C}\ \text{ft/s} \tag{6}$$

$$q = 0 \qquad \text{for} \qquad U_1 < U_{1C}\ \text{ft/s} \tag{7}$$

where q is the transport rate in cu yd/ft/yr, and U_1 is the wind speed in ft/sec at 3.3-ft (1-m) elevation. The quantity U_{1C}, which was determined to be 8.93 ft/sec from the present field study, is the threshold wind speed at the elevation of 3.3 ft.

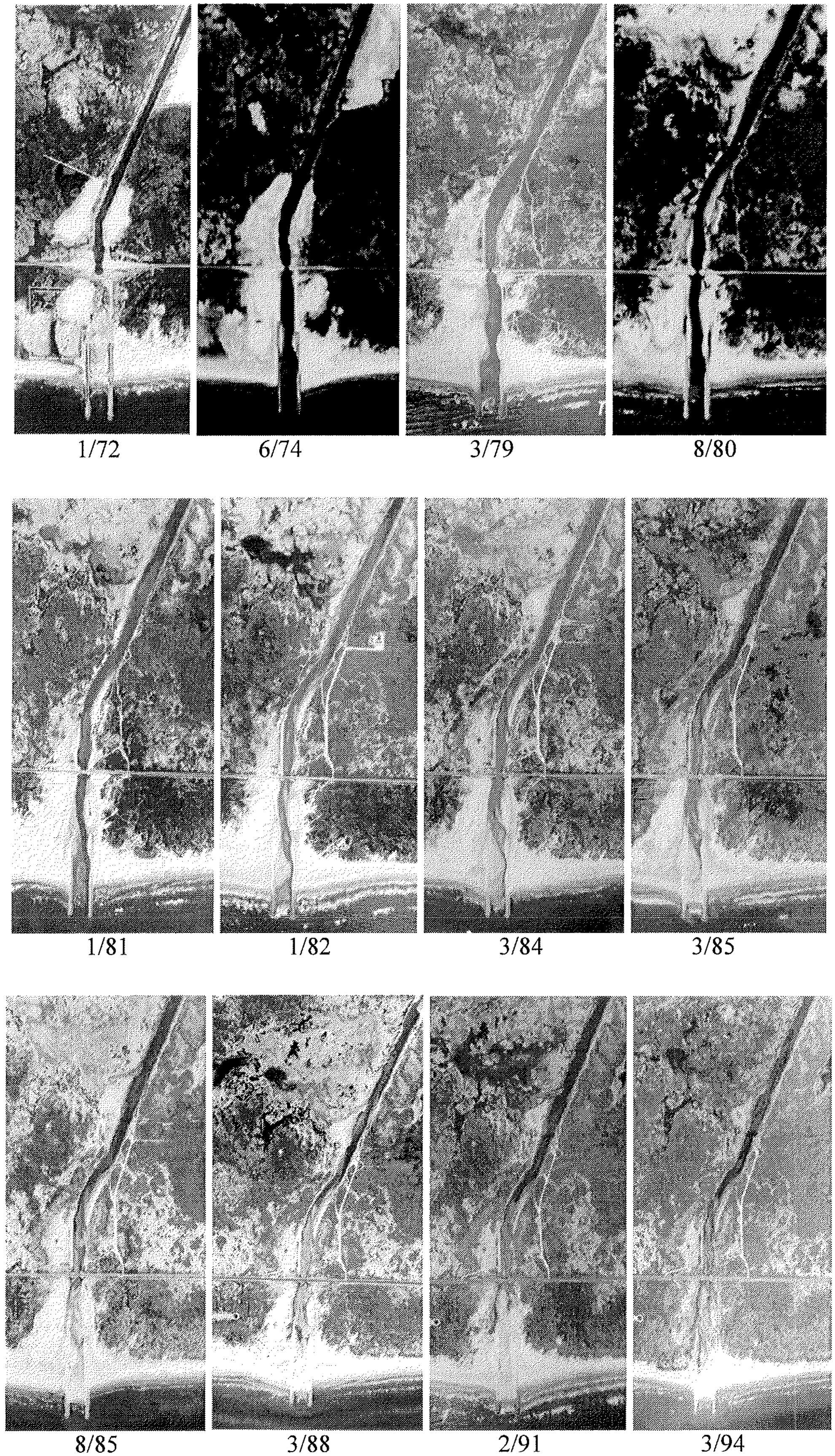

Figure 21. Aerial photographs of the Mustang Island Fish Pass (approximately 5 miles NE of Packery Channel) (Lanmon Aerial Photography, Inc. and USACE, Galveston). Note the influence of Hurricane Allen in August, 1980.

The average of 24 transport rates measured using the trap during the field study was 65 cu yd/ft/yr. To estimate the volume of infilling that would occur at the proposed channel, wind data for the period 1984 to 1993 collected by NWS (discussed in a previous section in this chapter) were input to Eq. (6). The NWS wind data, which were collected at an elevation of 17 ft, were transferred to equivalent speeds which would occur at a 3.3-ft elevation by using an empirical formula (1/7 power). A linear regression analysis was performed between measured wind speeds at 3.3 and 17 ft and showed a reliable correlation for the two elevations (giving a correlation coefficient of 0.967). These data were binned using a speed increment of 9 mph and direction increment of 22.5 deg. The mid-point direction and speed of each bin represented the wind vectors. The sine of the angle between each average yearly wind vector (see Figure 11) and the design channel orientation were calculated to determine the contribution of each vector to channel infilling. After separating the resultant vectors into two groups based on direction (i.e., groups contributing to infilling from north and south of the channel), the following equation was developed:

$$Q = 8.0\ SL + 18.1\ NL \tag{8}$$

where

Q = total flow of sand into channel (cu yd/yr)

SL = length of channel along south side of pass that is open to wind-blown sand transport (ft)

NL = length of channel along north side of pass that is open to wind-blown sand transport (ft)

In absence of preventative design considerations, the large volumes of wind-blown sand will probably cause a significant burden to maintenance dredging and may become one of the major contributions to channel closing. Based on the NWS wind data, wind speeds exceed the threshold wind speed 75% of the year. According to Eq. (8), approximately 70% of the infilling would occur from north of the channel, assuming equal lengths of open area along either bank. This bias from the north is because the wind from the south is predominantly from the southeast and is aimed along the channel, whereas the wind from the north is predominantly aimed directly at (perpendicular to) the channel. Eq. (8) does not consider the water content of the sand. Strong winds from the north are often accompanied by rain, which would reduce the potential for heavy wind-blown sand transport into the channel. Assuming 1,000 ft of open beach adjacent to both the north and south channel banks, Eq. (8) predicts that approximately 26,000 cu yd of sand would be blown into the channel per year if not controlled. Wind-blown sand counter-measures are discussed in Appendix F.

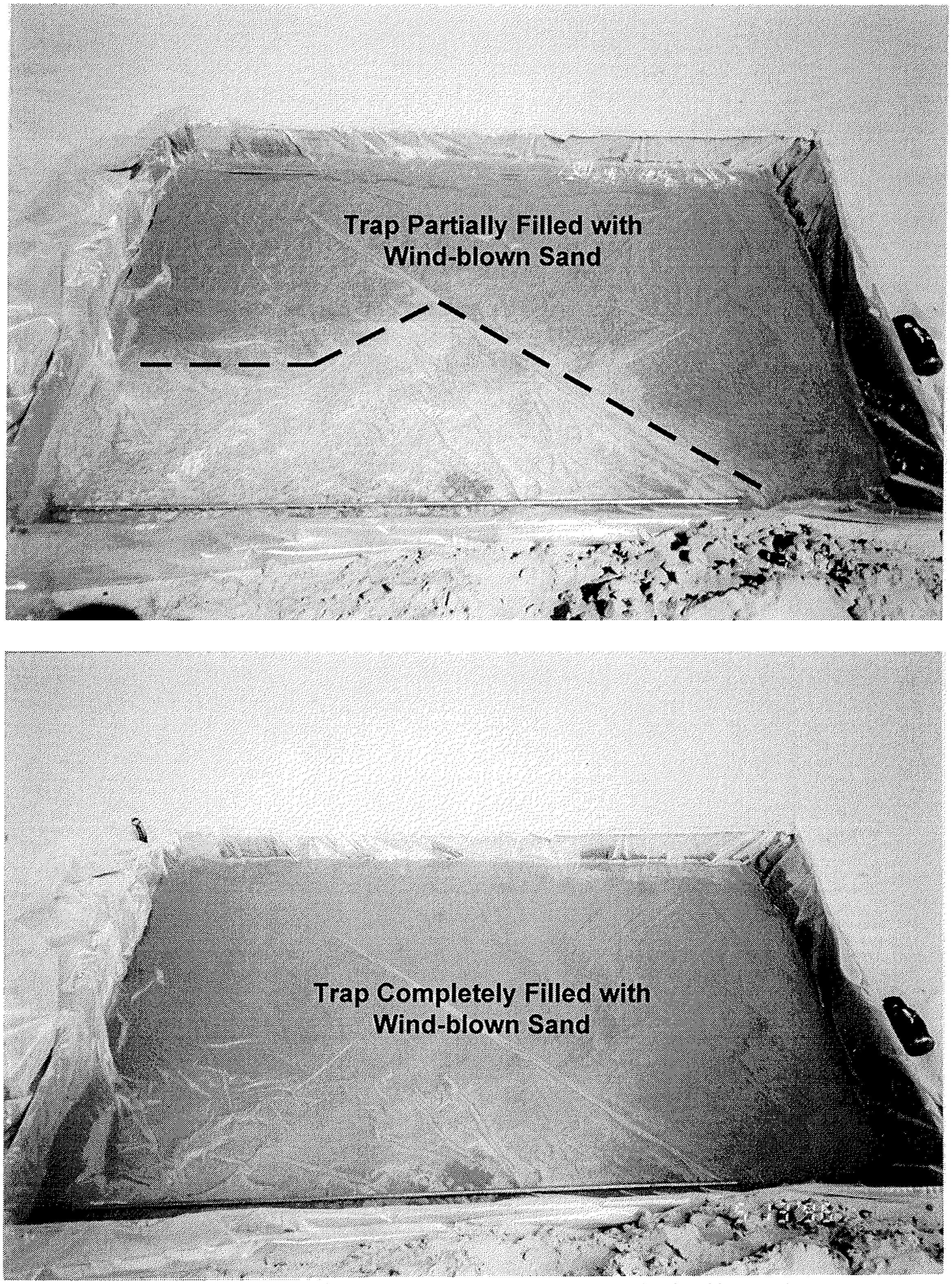

Figure 22. Field demonstration of sand trapping by an open water body. Upper photo represents 20 min and lower photo represents 40 min after trap installation.

3. Sand Management and Sediment Budget

This chapter summarizes sand transport mechanisms acting within the study area and considers the interaction of the proposed project with these mechanisms. A numerical model that simulates shoreline change is applied to the project area, a quantitative sediment budget developed, a functional jetty design presented, and sand management considerations discussed.

Methodology

Various components of the sediment budget are quantified through field data collection, review of previous studies, and shoreline-change modeling. After determining the key components of the sediment budget, project-specific sand management strategies and a jetty functional design are developed. The calibrated shoreline change model is then applied to examine alternative jetty design and develop a sand-management plan for the beaches adjacent to the proposed Packery Channel. This plan targets downdrift impacts of the proposed jetties and protection of the beach fronting the Padre Island seawall (which is presently threatened by a receding shoreline), and addresses such issues as channel maintenance dredging, sand bypassing volume and frequency, tidal shoal formation, and beach-fill volume and location of placement.

Sediment Budget

Beach change is produced by a difference in rates of inflow and outflow of sediment in the area under consideration. Sediment (assumed to be mainly sand from this point on) can enter or leave the area from the offshore, lateral ends of the beach, and the onshore. At the entrance of a newly cut inlet, the tidal current will tend to move sediment seaward to form an ebb shoal or bayward to form a flood shoal, thus removing sediment from the littoral transport system until the shoals reach equilibrium volumes. Sand management requires quantitative knowledge of these inflows and outflows to develop a plan for systematic preservation or improvement of the beach, particularly if engineering structures which may interrupt these flows are to be introduced to an area. The sediment budget refers to the quantity of sediment in the study area and the balance of sediment introduced, stored, or removed from the area, and it is an effective resource for considering sand-management strategies.

A coastal processes assessment was performed and a sediment budget developed for the Gulf shoreline fronting segments of north Padre and Mustang Islands within the proposed project area. The assessment was based on the following tasks:

1. Historical shoreline change data provided by the University of Texas, Bureau of Economic Geology (UT-BEG) were analyzed.

2. The present state of the beach and shoreface configurations was established based upon beach profile survey data collected by the Blucher Institute in April, 1996, and aerial photography.

3. Nearshore sediment movement representative of the project area was quantified through use of dredging records obtained from the USACE and through wave hindcast data.

4. Wind characteristics and wind-blown sand transport rates representative of the project area were quantified through analysis of wind data obtained from the NWS and results of a sand-trapping study conducted by the Blucher Institute.

5. An estimate was made of the volume of the ebb shoal that might be created from the entrance channel and adjacent beach sediments.

Results from these tasks were compiled to develop a local sediment budget for the project area as depicted in Figure 23. The dashed lines define the project area within which a control volume for the sediment budget was established.

The sediment budget accounts for the inflow and outflow of sand across boundaries of the project area. This bounded beach area is defined as the 6.1-mile (32,000-ft) stretch of shoreline extending from the Mustang Island Fish Pass to the southern end of the Padre Island seawall.

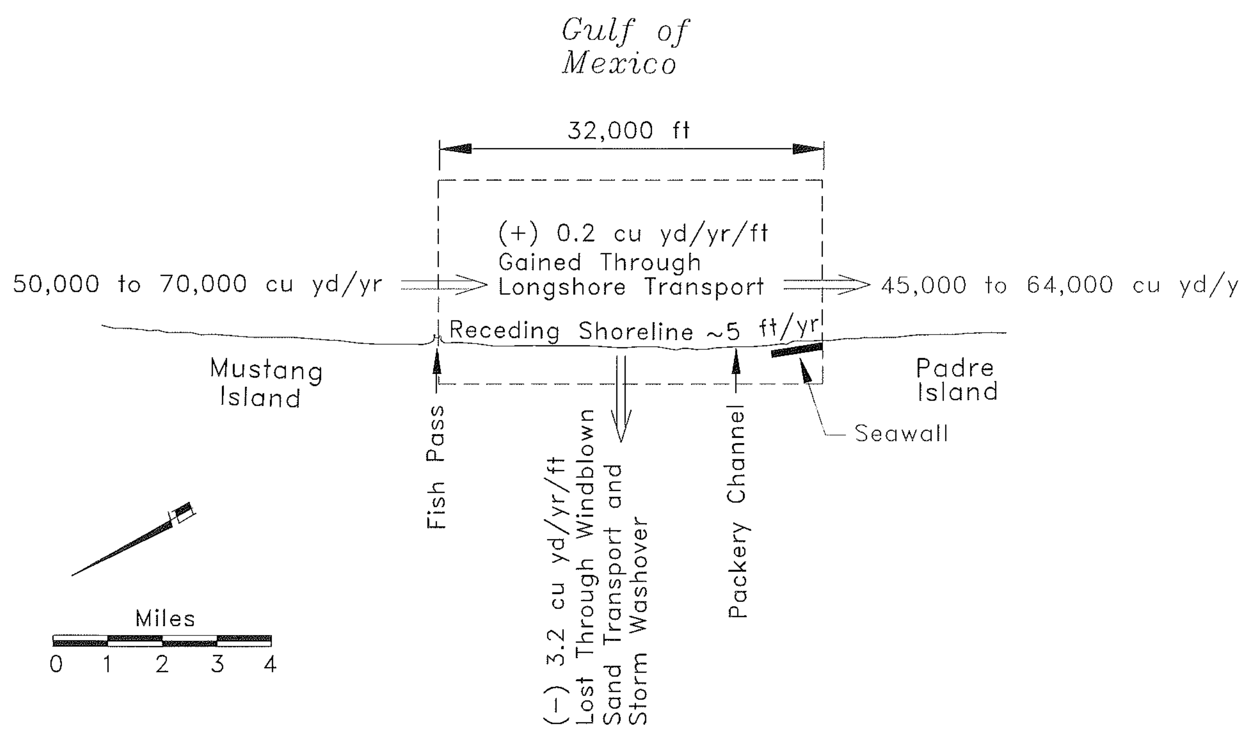

Figure 23. Sediment budget for beaches in Packery Channel vicinity.

The shoreline recession that has historically occurred along this bounded stretch has been well documented (Morton and Pieper 1977, Paine and Morton 1989). The developed sediment budget allows quantification of the flow of sediment that produces shoreline change and provides a means for predicting local shoreline evolution in a long-term (order of decades) framework.

Longshore Sand Transport

Evidence pertaining to the regional, long-term net direction of sediment transport along Mustang and north Padre Islands suggests that net transport is directed to the southwest (see Chapter 2), although annual reversals may occur. As summarized in Weise and White (1980), a zone of long-term net longshore drift convergence exists approximately 50 miles to the southwest of Packery Channel near 27 deg north latitude. The existence of this convergence zone and the generally consistent concave shape of the coastline indicate that the long-term net longshore transport rate decreases regionally from northeast to southwest through the study area.

To calculate the local alongshore variability of longshore transport, the OCTI wave hindcast data for the years 1970 to 1979 (excluding the years 1972 and 1976) and June, 1974, digitized shoreline position data (obtained from UT-BEG) were formatted and input into the numerical shoreline change numerical simulation model GENESIS (Hanson and Kraus 1989). This shoreline change model is supported by the USACE and installed at more than 500 registered sites worldwide. The model calculates shoreline change under a wide range of user-specified boundary conditions, wave conditions, coastal structure configurations, beach fills, and other processes and engineering designs. The value used for the dimensionless proportionality coefficient K in the model (which relates transport rate to the longshore component of wave energy flux) was adjusted to obtain upper and lower limits of sediment transport rates. Typical values of K vary between 0.58 (Kraus *et al.* 1982) and 0.77 (Komar and Inman 1970) for root-mean square wave height.

As shown in Figure 24, the regional trend is for longshore transport to decrease from north to south along the study area Gulf shoreline. Note that the gross transport rate remains relatively constant as a function of distance alongshore. Based on an estimated gross transport rate at Port Mansfield of about 175,000 ± 126,000 cu yd/yr, the range of gross transport rates of approximately 175,000 to 250,000 cu yd/yr calculated using the hindcast data is considered a realistic upper envelope. Note that this estimate of gross longshore sediment transport is considerably lower than the estimate of Watson and Behrens (1976) of 730,000 cu yd/yr discussed in Chapter 2. The calculated average net longshore transport rate near the Fish Pass is approximately 50,000 to 70,000 cu yd/yr. This range is consistent with a 3-year average calculated by Watson and Behrens (1976) of 66,600 cu yd/yr. The net rate at the south end of the seawall varies from 45,000 to 64,000 cu yd/yr.

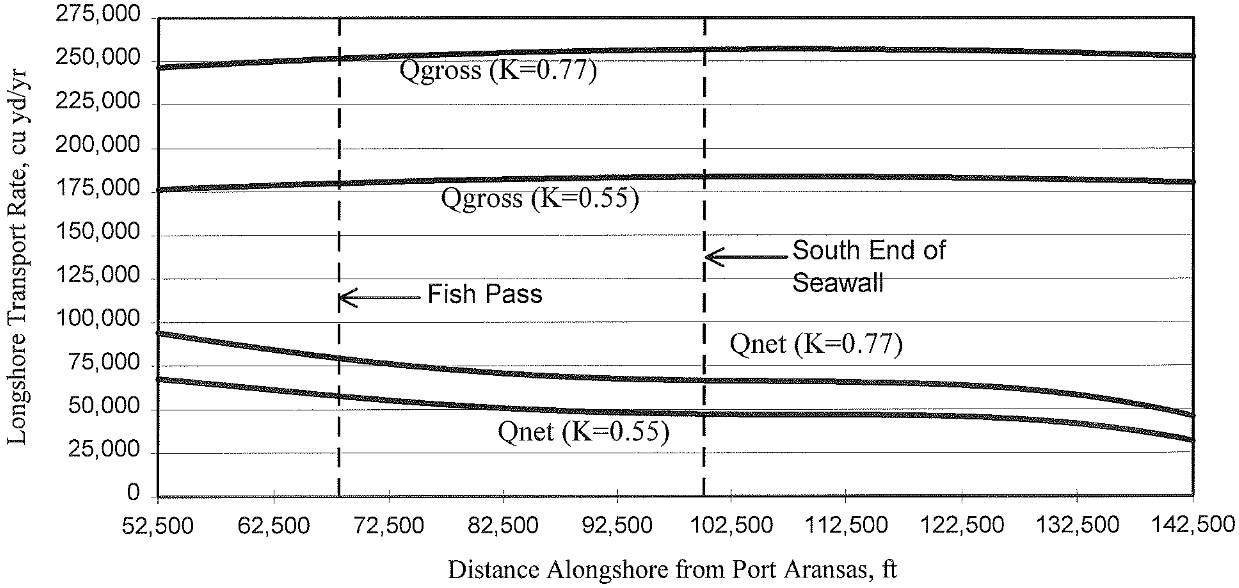

Figure 24. Variation in average longshore transport rate along study area Gulf shoreline.

By balancing the net longshore inflow and outflow of sand transported through the study area, an average deposition of approximately 0.2 cu yd/yr/ft was calculated along the 32,000 ft bounded beach stretch. Because the study area experiences beach erosion despite the apparent net contribution of sand by longshore transport, other transport processes significantly contribute to the sediment budget.

Wind-Blown Sand

As discussed by Shepard and Rusnak (1957), the persistent southeast winds along north Padre and Mustang Islands are a significant agent in moving sediment across the barrier islands. Aided by the arid climate, which produces relatively minimal vegetation, considerable volumes of sand are transported towards the Laguna Madre and Corpus Christi Bay and can contribute to shoreline advance along the west sides of the islands (Morton and Paine 1984). Weise and White (1980) documented dune migration rates of approximately 35 ft/yr across Padre Island, and cited other studies (Hunter and Dickenson 1970, Price 1971) which measured migration on north Padre Island of up to 75 and 85 ft/yr. Inspection of the 1984 to 1993 NWS wind data reveals that onshore winds with speeds greater than 9 mph occur 58% of the year, implying that substantial sand will be transported landward by wind for about half the year, on average. As observed by Hayes (1965), Berryhill (1969), Suter *et al.* (1982), and Dahl *et al.* (1983), cross-shore sand transport also occurs regionally on a large scale during storm surges that accompany hurricanes, which deposit sand through low-lying breaches in the foredune system. The rate of cross-shore transport by wind probably achieves a maximum near the longshore convergence zone to the south of the study area, where fine sediments are transported inland to the wind-tidal flat infill of the Laguna Madre (the "Land Cut," which separates the Lower and the Upper Laguna Madre) (Hayes 1965).

The cross-shore sediment transport rate was back-calculated as part of the sediment budget to provide a balance between the sand gained through longshore transport and the sand that is lost during shoreline recession. In this calculation, it was assumed that net transport across the Gulfward (offshore) boundary was zero. The long-term rate of shoreline recession was generalized for the bounded beach stretch as 5 ft/yr based on shoreline change rates as discussed in Chapter 2. Relative sea-level rise, which contributes about a 1 ft/yr to the shoreline recession, is not associated with a loss in beach volume. Therefore, a recession rate of 4 ft/yr is used with an equilibrium beach profile shape, which assumes the shape of the beach profile remains constant over time (see Dean 1991), to calculate a beach-volume loss of 3 cu yd/yr/ft. This volume change rate was calculated by multiplying the vertical distance from the berm crest to the closure depth (this distance was approximated here as 20 ft) by the rate of shoreline change and by a unit distance alongshore. The total rate of cross-shore sand loss was then calculated based on the difference between the 0.2 cu yd/yr/ft gain from longshore transport and the 3 cu yd/yr/ft loss associated with shoreline recession as 3.2 cu yd/yr/ft. The sediment budget therefore reveals that within the control volume of the project site (as depicted in Figure 23), a gradient in net longshore transport gives a relatively minor contribution to the sediment budget (see also Capodice 1985); shoreline recession and beach loss are dominated by cross-shore processes (i.e., landward wind-blown transport and storm washover).

A calculation was made to estimate potential wind-blown sand transport across the island and to emphasize the dominance of cross-shore transport over longshore transport on local beach erosion. The calculation, which is consistent with the method used to predict channel trapping of wind-blown sand in Chapter 2, is based on the 1984 to 1993 wind data collected by NWS (see the wind rose in Figure 9) and the empirical wind-blown sand-transport equation, Eq. (6). This interpretation of potential wind-blown sand transport across the island includes the assumption that the sand is always dry and is not held in place by vegetation or structures (e.g., buildings, bulkheads, the seawall, etc.). Keeping in mind that the island is eroding despite a longshore transport gradient which contributes sand to the control volume and that the onshore component of wind dominates, the following results represent a net landward transport of wind-blown sand across the control volume:

$$q_{\text{landward}} = 29.2 \text{ cu yd/ft/yr}$$

$$q_{\text{seaward}} = 8.1 \text{ cu yd/ft/yr}$$

$$q_{\text{net}} = 21.1 \text{ cu yd/ft/yr (landward)}$$

Note that this rate is considerably higher than the rate of 3.2 cu yd/ft/yr which was back-calculated out of the sediment budget. However, the lower rate includes sand impoundment by vegetation and structures, as well as reduction in transport by moist sand. The lower rate also accounts for storm washover, which occurs in both the landward and seaward directions. For example, seaward washover, or flooding, occurred after the September, 1961, Hurricane Carla (Hayes 1965) and after the September, 1967, Hurricane Beulah (USACE 1968) when strong currents generated by the ebb of the storm-surge tide transported sediment towards the beach. The calculated potential cross-shore wind-blown transport rate of 21.1 cu yd/ft/yr should be considered as an upper limit and is included to emphasize the importance of wind-blown sand in the sediment budget for the study area.

Tidal Shoals

The tidal current at a newly cut inlet will tend to create an ebb-tidal shoal and a flood-tidal shoal. The shoals are formed from sediments scoured from the bottom and banks of the channel by the tidal current, and from sediments transported along the beach that fall into the channel. These sediments must be accounted for in a sediment budget aimed at estimating shoreline change.

A flood shoal will not form at Packery Channel because the deposition basin near the SH361 bridge will collect sediment brought to it. This sediment should be dredged from the deposition basin and placed on the adjacent beaches as part of the sand-management plan. Some fine sediments scoured from the northern end of the channel may collect at its entrance in the Laguna Madre. This material would not come from the Gulf beaches and is not considered further in the sediment budget.

Formation of an ebb-tidal shoal must be included in developing a sediment budget for the region of Packery Channel. Morton (1977b) documented changes in volume of the ebb-tidal shoal and adjacent beaches for seven jettied inlets on the Texas coast. One of these was Mansfield Pass, discussed in detail in Chapter 2 because of its proximity to the project site. The ebb current at Mansfield Pass appears to be stronger than the flood current (in most seasons, based on limited measurements), yet no significant ebb-tidal shoal has formed at its entrance. Packery Channel is expected to have a tidal distortion in which the flood current is stronger than the ebb current, based on computer simulation (Brown and Militello 1997) and by analogy to the Mustang Island Fish Pass (Watson and Behrens 1976, Behrens *et al.* 1977). Flood dominance implies a relatively small ebb-tidal shoal. In addition, if the channel is dredged and sand accumulated on the beach bypassed by artificial means, then an ebb-tidal shoal will not form. Although development of a substantial ebb-tidal shoal at the entrance of Packery Channel appears doubtful, it is worthwhile to estimate the potential volume that might be contained in it.

An empirical procedure for estimating the equilibrium volume of an ebb-tidal shoal was developed by Walton and Adams (1976) based on analysis of 44 tidal inlets, including Aransas Pass, Texas, and Galveston Entrance, Texas. They divided the coasts of the United States into three categories according to incident wave climate. The Texas coast was classified into the "mildly exposed coasts" category, as opposed to "highly exposed" and "moderately exposed" coasts, indicative of the smaller wave heights and shorter wave periods for the Texas coast as compared to other coasts. The average significant wave height was estimated by Walton and Adams to be 1- to 1.5-ft and the wave period 4 sec for the Texas coast. Smaller waves would have less potential to move sand back to the beach that is brought to an ebb-tidal shoal by the tidal current. Therefore, ebb-tidal shoal volumes are, in principle, expected to be greater for mildly exposed coasts, all other factors being equal, as found by Walton and Adams.

The equilibrium ebb-tidal shoal volume is computed as

$$\cancel{V} = \alpha P^{1.23} \tag{9}$$

in which the volume $\cancel{V}$ is given in cubic yards, P is the tidal prism given in cubic feet, and α is an empirical coefficient found to have the value 13.8×10^{-5} for mildly exposed coasts. The tidal prism is discussed in Chapter 4 and was estimated to be 6.52×10^{7} cu ft, although this value may be an underestimate. This value is slightly less than the smallest value (for Venice Inlet, Florida) in the Walton and Adams (1976) database. Eq. 9 gives $\cancel{V} = 560{,}000$ cu yd for the proposed Packery Channel. If this shoal forms, a portion of the material will come from the adjacent beaches. Therefore, as part of the project monitoring plan, detection of the ebb-tidal shoal should be included. Development of the ebb-tidal shoal, if it occurs, would take place over several years and can be reduced by construction of jetty spurs (discussed below) and by channel maintenance dredging and sand bypassing.

Beach Fill and Sand Bypassing

The interruption of the natural littoral system along Mustang and north Padre Islands which will result from introduction of the jettied Packery Channel must be balanced through artificial means as determined through analysis of a site-specific sand management strategy. Beach fill and sand bypassing were identified as two key components of this strategy. Applicable beach regulations and sand-management guidelines are presented in City of Corpus Christi (1995a, 1995b), Nueces County (1995), and the Texas General Land Office (see State of Texas 1993, §§15.7(d) of the Texas Administrative Code, *Requirements for Beach Nourishment Projects*).

Beach Fill

Beach fill is the artificial placement of material from an external source onto the beach, and the process is referred to as beach nourishment. Beach nourishment is popular on the east coast of the United States, in particular, Florida, which has a billion-dollar tourist industry associated with its beaches. In Texas, only three major beach fill projects have been conducted. In 1978, the first major beach fill was placed on Corpus Christi Beach (North Beach) in Corpus Christi Bay (the project concept was originally studied by W. Armstrong Price in 1956), which involved federal participation (Kieslich and Brunt 1989). The Corpus Christi Beach fill is termed a "veneer fill" because it followed an innovative procedure of placing a layer of fine-grained dredged material on the eroding shore which was then covered with a protective layer of medium-grained sand. The second major fill was placed from Autumn, 1994, through Spring, 1995, in front of the Galveston seawall facing the Gulf of Mexico (McKenna *et al.* 1995). This material was pumped on the beach from an offshore sand source. A third beach fill was placed in February, 1997, along the eroding Gulf beaches at the Town of South Padre Island using beach-quality sand dredged by the USACE from the Brazos-Santiago Entrance Channel (Heilman and Kraus 1995).

Pumping of beach-quality material removed during new-work and maintenance dredging operations at Packery Channel onto the beach downdrift (south) of the proposed jetties is expected to be an economical means of establishing beach width. Based on the permitted channel dimensions (Appendix A) and required dredged material volume estimates presented in Goldston Engineering, Inc. (1985b) and in Shiner, Moseley and Associates, Inc. (1987), at least 750,000 cu yd of sediment will be available after initial dredging of the channel (the excavation of a boat basin was included in this calculation). According to Morton and Pieper (1977), studies have been conducted which indicate the presence of a 45- to 50-ft deep layer of sand under southern Mustang Island (in the vicinity of Packery Channel) and a 60-ft deep layer of sand under north Padre Island. Therefore, the present study assumes that at least 500,000 cu yd of beach-quality sand will be available for placement along the Gulf beach south of the channel. This beach fill will delay the impacts of shoreline recession downdrift of the jetties and will provide immediate protection to the seawall and south jetty should a storm arrive prior to the post-project beaches reaching stable configuration. Remaining dredged sand might be stockpiled to respond to emergencies (storms) or supplement the original fill. Further discussion of the recommended beach fill dimensions and location is presented later in this chapter. For general guidelines on the design of beach fills see USACE (1995b).

Sand Bypassing

Sand bypassing refers to the transfer of littoral sediment past a natural or engineered discontinuity or obstruction along a coastline. In many cases, this bypassing occurs naturally through a number of different mechanisms involving bars, shoals, and/or tidal currents. Any coastal construction project that impounds or diverts littoral sediment creates the potential need for artificial sand bypassing, or the mechanical transfer of sand past the project in the direction of net longshore sediment transport, to prevent erosion from occurring downdrift of the project and to minimize channel shoaling. The volume of sand bypassed and the bypassing frequency are determined considering a number of factors, including: 1) the storage capacity of the updrift sand impoundment area, 2) the net rate of longshore drift, and 3) the cost of the bypassing operation.

Sand bypassing system selection is outlined by USACE (1991) and in Richardson (1991), and a design review was conducted for the project site by Duke (1985b). Further discussion of the recommended sand bypassing plan is presented later in this chapter.

Jetty Functional Design

Jetties are constructed to maintain the design depth of an inlet channel and reduce maintenance dredging by concentrating and directing currents to increase channel scouring action. Another function of jetties is to afford sheltering to allow vessels to cross the surf zone without encountering breaking waves, cross waves, and cross currents. Jetties stabilize a channel entrance by blocking the longshore drift and minimizing sediment deposition in the inlet channel. Because jetties block longshore drift, they alter natural sediment transport paths and may promote downdrift beach erosion or removal of sand from the littoral system. Morton (1977b) found that local accretion can occur on both sides of jetties along the Texas coast, as well as updrift accretion and downdrift erosion, the situation expected if no ebb-tidal shoal forms. Construction of jetties should include provisions for mitigation of any significant alterations to the natural transport processes. An overview of jetty design and theory is presented in USACE (1995a), and a review of conceptual jetty design alternatives for Packery Channel is presented in Duke (1985b). Komar *et al.* (1976) investigated shoreline impacts at jetties constructed on the Oregon coast in areas of near-zero net longshore sediment transport, similar to the situation for Packery Channel. They found that considerable shoreline change can result from jetty construction, even in an area of low net sand transport. However, as opposed to the situation where jetties block appreciable sand (non-zero net), an equilibrium shoreline configuration was reached within a few years.

In 1962, Carothers and Innis presented design concepts for stabilization of Corpus Christi Pass including channel armoring and jetties extending to the 6-ft depth contour. The recommended jetties were curved outward at their Gulfward ends and aligned approximately 15 deg south of shore-normal to be parallel with the prevailing winds. This channel alignment was intended to minimize channel intrusion by wind-blown sand and to improve water interchange. In 1967 (with a revision in 1970), Turner, Collie and Braden, Inc., presented a design for New Corpus Christi Pass (now known as Mustang Island Fish Pass), recommending 1,400-ft long jetties spaced 400 ft apart, a 10-ft MLW entrance channel depth, and annual maintenance dredging. The Fish Pass design was based on a gross longshore transport rate on the order of 250,000 cu yd/yr as estimated by Lockwood, Andrews and Newnam, Inc. (1959).

Jetty Length and Width

The permitted jetty design for Packery Channel (as shown in Appendix A) allows for the construction of dual jetties extending a maximum length of 1,500 ft into the Gulf. The permitted channel is aligned at approximately 12 deg north of shore-normal, has a bottom width of 140 ft at a depth of 10 ft MSL, and has 1 on 3 side slopes. The functional jetty design presented in this report operates within general parameters of the permitted design.

Because the majority of sand that moves alongshore is transported within the surf zone and because one purpose of jetties is to prevent this flow of sand from reaching the channel, a key consideration in jetty design is the depth contour to which the jetties should extend. It has been well recognized that the jetties constructed at the Mustang Island Fish Pass, which extended only 870 ft into the Gulf (approximate 8-ft depth contour MSL), did not traverse the outer bar and thereby allowed sand to be easily transported around their seaward ends and into the channel. Based on beach profiles surveyed at the Fish Pass (Watson and Behrens 1976 and Behrens *et al.* 1977), Duke (1985b) estimated that jetties at Packery Channel should extend at least 1,500 ft offshore to intersect the 10-ft depth contour, and recommended a jetty length of 1,800 ft to minimize shoaling at the jetty entrance.

Jetty length at Packery Channel was determined in the present study through inspection of the beach profile survey data collected in April, 1996, and historic profile survey data (see Watson and Behrens 1976 and Behrens *et al.* 1977). The criterion was to minimize expected natural sand bypassing around the ends of the jetties with the minimum length of jetty. To achieve this objective, it was determined that the seaward ends of the jetties should be placed at a depth that is located seaward of the most seaward bar, the "storm" bar. Navigation charts for all coasts of the United States are being converted to the mean lower low water (MLLW) datum, and depths from the permitted design were re-interpreted to the MLLW datum. In order to assure 10 ft of navigable depth to MLLW and that the jetties traverse the outer bar, the jetties must extend at least 1,400 offshore measured from the present location of the MSL shoreline, as shown schematically in Figure 25. At the project site, the MLLW datum lies approximately 1 ft below MSL and MTL (Figure 13). In developing a design for the Mustang Island Fish Pass, Turner, Collie & Braden, Inc. (1967a, 1970) compared hypothetical total-cost-per-navigable-year figures for jetty lengths from 1,000 to 2,200 ft, and also recommended a jetty length of approximately 1,400 ft. For a reference, the Bob Hall Pier (Figure 1) extends 1,050 ft from the approximate 1996 MSL shoreline.

Only larger waves will break on the outer bar, and recreational (and other) vessels are not expected to be (and should be discouraged from being) underway in Packery Channel during rough conditions. Therefore, if the entrance channel is maintained to the recommended design depth of 10 ft MLLW, the jetties will provide safe passage through the surf zone, and no depth-limited breaking waves will appear in the channel to endanger navigation. Because MLLW is a relatively extreme low water datum, as shown in Figure 14, typically there will be an extra 1 or 2 ft of water above chart datum, and only occasionally will the water level fall below chart datum, and then for only short periods of time. Jetty width of 300 ft with a 140-ft channel bottom (as specified in the design permit) satisfies accepted safety criteria for recreational vessel usage (American Society of Civil Engineers 1994). This width also gives a favorable entrance length-to-width ratio for stability, as discussed in the next chapter.

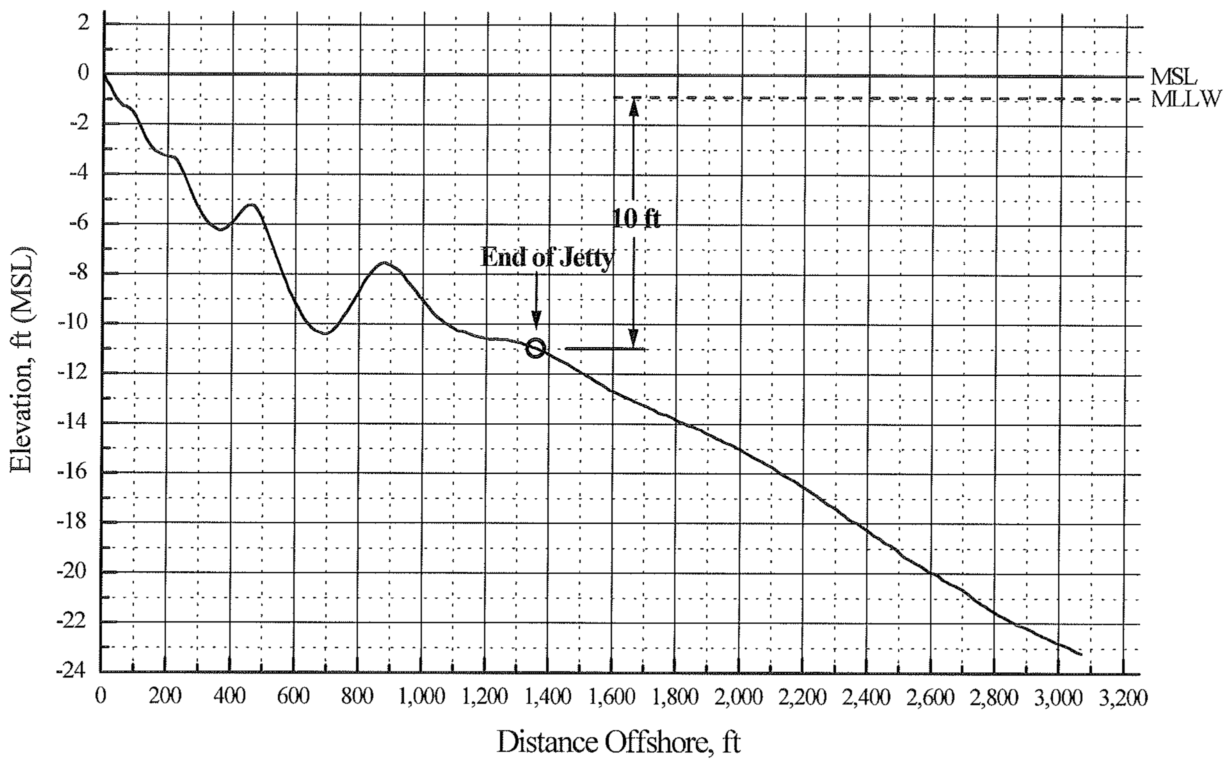

Figure 25. Minimum depth contour to which jetties should extend (average beach profile shown).

Jetty Location and Orientation

The next components of jetty design considered were jetty location and alignment. As noted by Price (1952) and Price and Parker (1979), natural inlets along the upper Texas coast (including Packery Channel) historically maintained nearly north-south orientations and southerly locations with respect to their bay systems. This tendency owes mainly to a contribution of strong northern winter winds which funnel flow from the bays and through the inlets, thereby promoting maximum flow and associated reduction of channel sedimentation. Packery Channel is considered an ideal location for an inlet because it is located south of Corpus Christi Bay and is near a convergence zone where the net longshore sand transport and associated downdrift impacts are relatively small.

Jetties are usually aligned parallel to the selected channel alignment to maintain channel flow velocities. At Packery Channel the permitted channel is aligned 12 deg north of shore-normal, or slightly north of ESE. Although the natural (stable) orientation of Packery Channel was north-south, the proposed alignment is along a slightly east-west former opening which was probably cut by a hurricane (Figure 26) and is the washover path that has dominated since closure of the inlet. Because of the existing infrastructure south of the proposed channel, it is recommended that the jetty and channel remain aligned as permitted. This alignment provides a direct route to the 11-ft MSL depth contour and minimizes required jetty length (see Figure 25). In addition, as shown in Figure 27, the 12-deg north of shore-normal alignment will afford sheltering from waves incident from the ESE and SE, which are predominant during the summer, the most active boating season.

Table 3 compares the geometries of the proposed Packery Channel and neighboring present and historical passes. The amount of dredging at each pass increases proportionally to the length and width of the channel. Also, as is shown in the next chapter, for a fixed channel depth, as the width of the inlet increases, the tide- and wind-induced current velocity decreases, thereby decreasing the self-scouring potential or capability of the inlet to maintain itself. The proposed design width to depth ratio of Packery Channel improves upon those of the Fish Pass and Mansfield Pass, implying that Packery Channel should be much more self maintaining (see Chapter 4).

Table 3. Approximate geometry of five Texas Gulf Coast passes.

Pass	Jetty Length, ft (perpendicular to shoreline)	Estimated Depth at Jetty Tip, ft	Distance Between Jetties, ft	Depth of Entrance Channel, ft	Width to Depth Ratio
Colorado River	East Jetty: 1,400 West Jetty: 600	13	1,300	16	81
Aransas Pass	North Jetty: 11,000 South Jetty: 9,000	30	1,300	47	28
Fish Pass	870	8	400	8, 4[9]	50, 100
Packery Channel (Proposed)	1,400	11	300	11	27
Mansfield Pass	1,350[10]	15	1,000	16	63

[9] Although the Fish Pass entrance channel was dredged to a depth of 11 ft, the channel shoaled to an 8-ft depth within 1 month and then to 4-ft depth several months thereafter.
[10] Measured from the 1962 shoreline position as existed during construction of the present jetties.

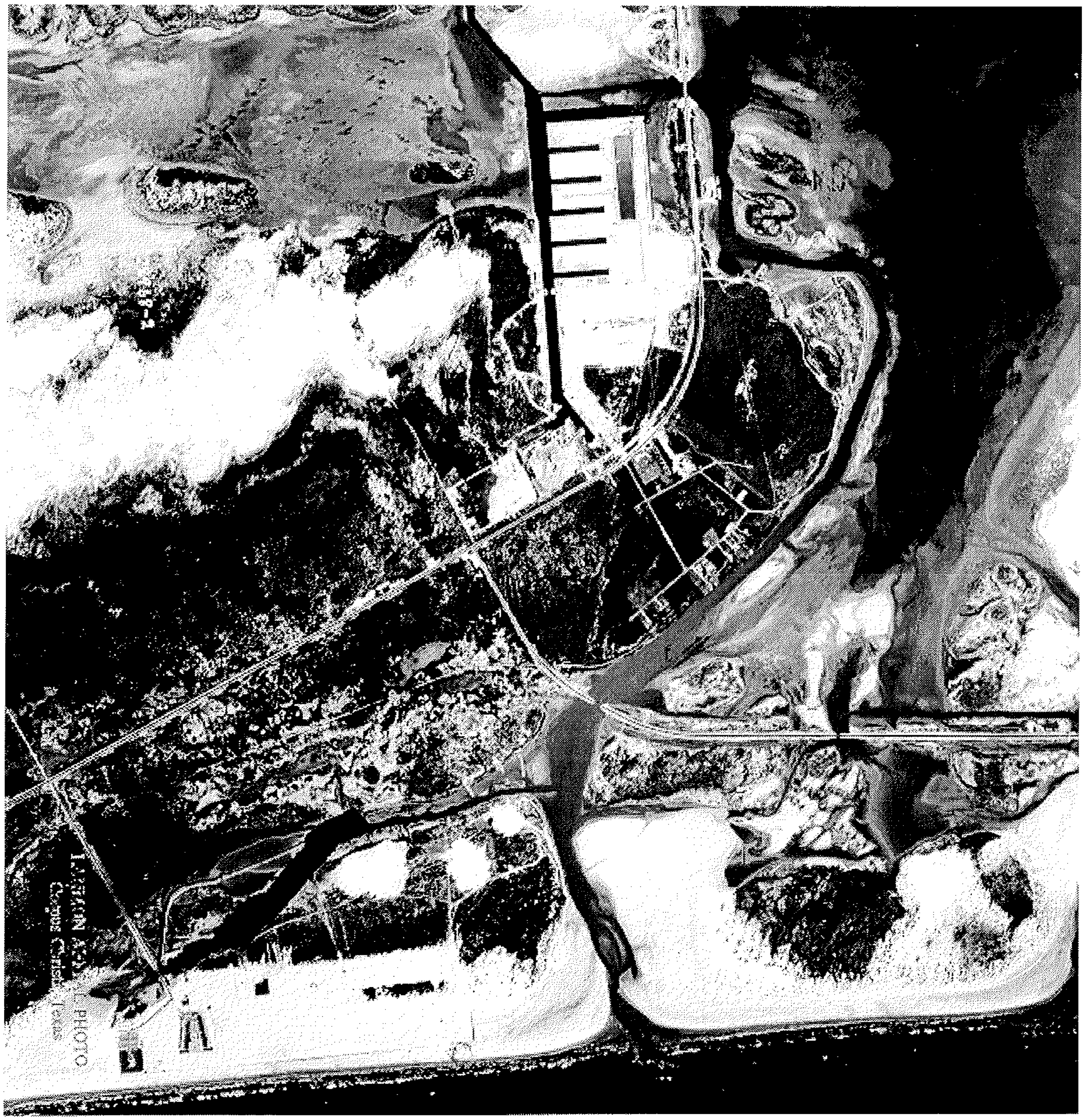

Figure 26. Packery Channel 14 months after September, 1967, Hurricane Beulah (photograph taken November 4, 1968, by Lanmon Aerial Photography, Inc.).

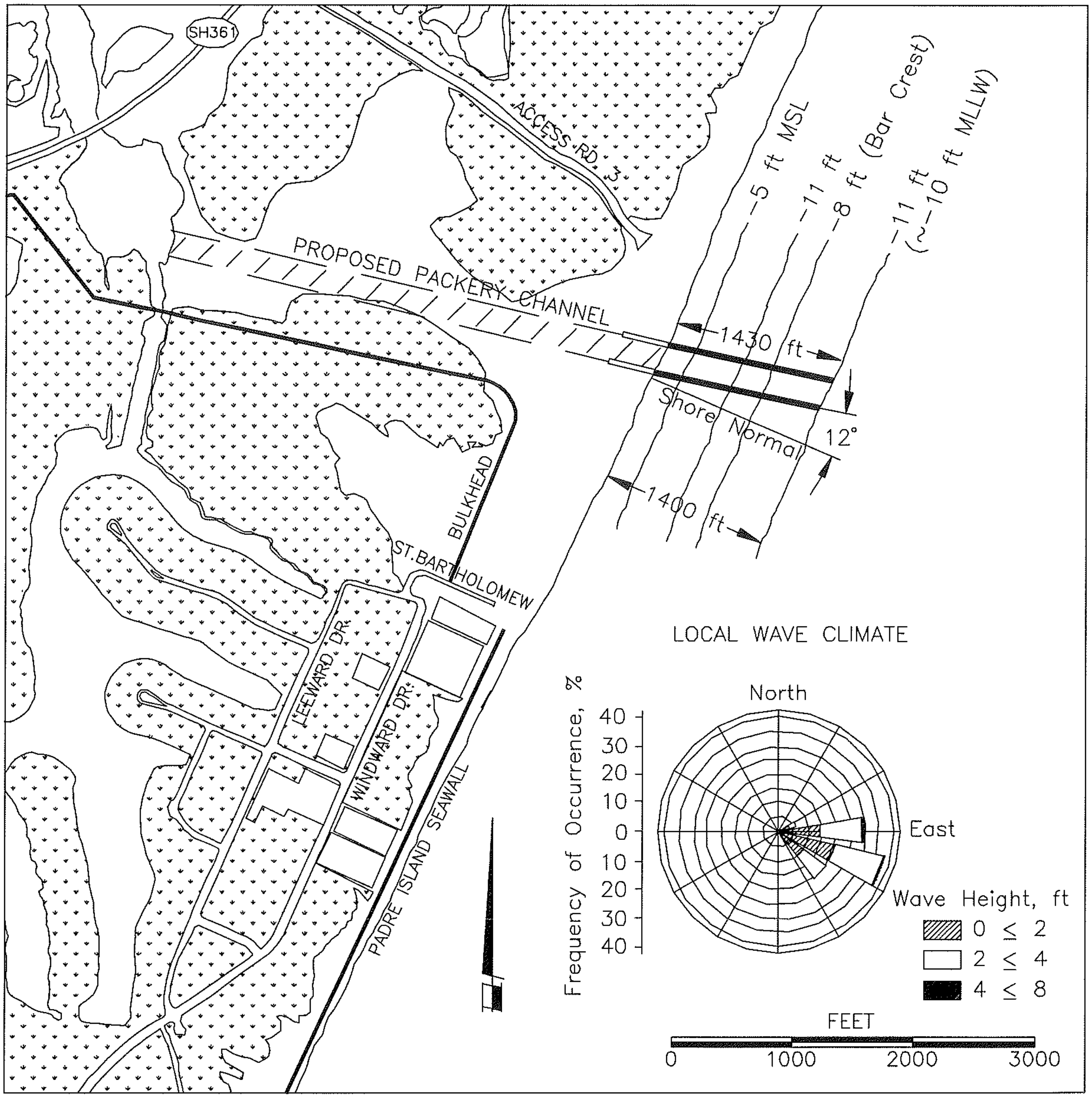

Figure 27. Proposed alignment of Packery Channel and recommended minimum jetty length.

Spur Jetties

Rip currents are narrow but strong seaward flows of water, and they typically form and persist on the sides of jetties, in particular, on the side blocking the longshore current. Rip currents pose a danger to weak or unknowing swimmers, but they are sought by surfers, who ride them through the surf zone. Rip currents also remove sand from the surf zone by jetting it offshore. In the overall horizontal circulation pattern, the mobilized sediment brought offshore in the rip can be transported around the jetty, as shown by the suspended sediment cloud in Figure 21, and then deposited in the channel. Even though the ends of the proposed jetties will initially extend some 1,400 ft from the MSL shoreline, with passage of time sand will be impounded or blocked by the jetties and a fillet will grow on the updrift side (and, perhaps, on both sides, as for the Fish Pass) (see also, Morton 1977b). The shoreline and other depth contours will correspondingly move out toward the end of the jetties, and sand will be more easily bypassed by currents.

Because an effective jetty design should minimize channel sedimentation by blocking longshore sediment transport while simultaneously minimizing adjacent beach impacts, the design should re-direct the flow of longshore drift so that sediment deposition occurs in a pre-determined area. The sand can then be dredged and bypassed periodically or on an as-need basis to the downdrift beach without compromising navigation safety caused by sediment shoaling in the channel entrance. For the Packery Channel jetties, it is proposed that emergent spurs be attached near their seaward ends (Figure 28) to deflect the longshore sediment flow away from the channel entrance and into a localized area, thereby allowing for efficient planning and phased operation of maintenance dredging.

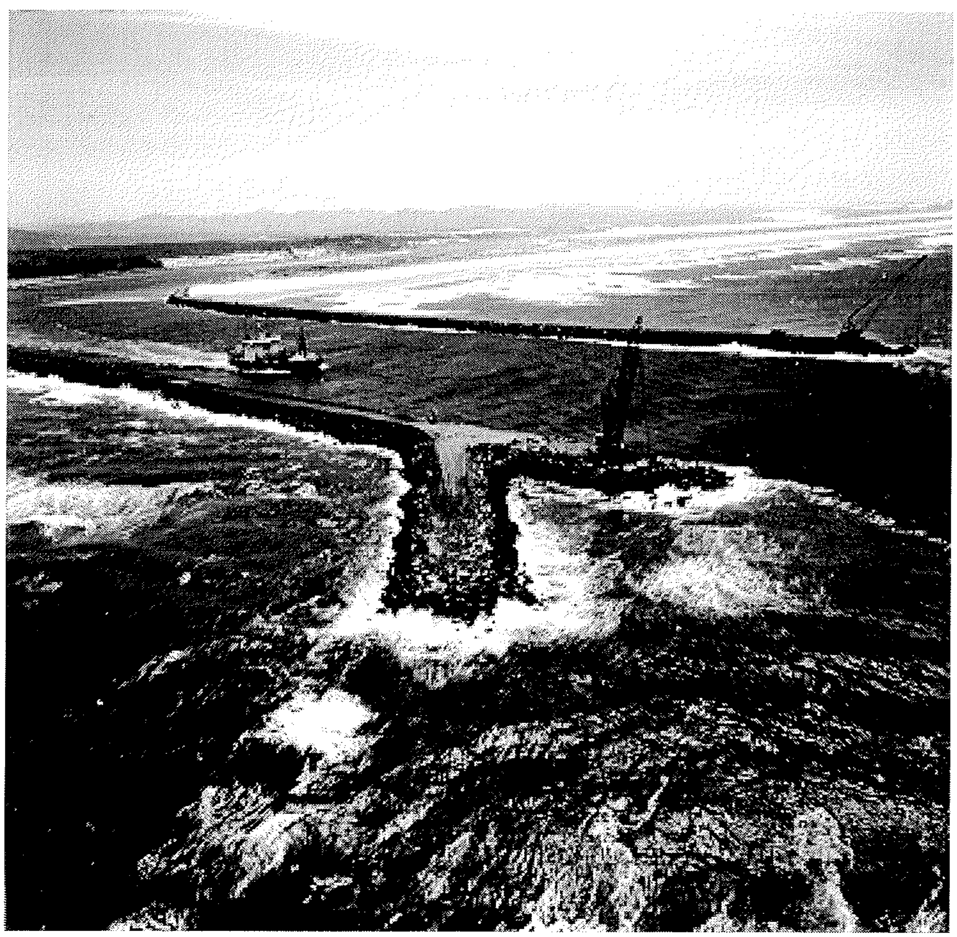

Figure 28. Construction of spur jetties at the entrance to the Siuslaw River, Oregon, 1985.

Spurs are sometimes placed inside of jetties to force the current away from the sides of the jetties. Rarely have they been placed outside of jetties as recommended here, but the function is similar – to force the (rip) current away from the end of the jetty. Spur jetties are expected to contain fillet growth and to direct the rip current and transported sediment away from the jetty, keeping the sediment in or near the surf zone.

Spur jetties have seen trials in Japan (Sasaki and Sakuramoto 1984)[11], and recently a pair of external spurs was placed on an extension of the jetties at Siuslaw River, Oregon (Pollock 1995) by the USACE. In 1985, the rubble-mound jetties at the entrance of the Siuslaw River (Figure 28) were extended offshore with spurs attached at a 45-deg angle "...to reduce sediment shoaling and dredging requirements in the navigation channel." The concept of the spurs at the Siuslaw jetties arose as an outcome of physical model studies for the Rogue River navigation project on the Oregon coast (Bottin 1982). Model results with the spur-jetty alternative indicated that sediment brought to the jetties by the longshore current was entrained in an eddy (probably the diffraction current created by the spurs, as well as deflection of the rip current), and the eddy tended to deflect the material away from the structure. Sediment was transported back toward shore where it was either reintroduced to the littoral system or carried by a jet of water away from the jetty and parallel to the spur. (In the present situation of Packery Channel, the sediment deflected away by the spurs might form a broad shoal offshore and serve as a surfing enhancement by altering breaking wave characteristics, although this hypothesis is speculative at present and was not investigated in this study.)

As depicted in Figure 29, the spurs will re-direct the longshore current and drift which are intercepted by the updrift jetty and may deposit the sand in a shoal away from the channel entrance. This process prevents sand from flowing around the jetty ends and into the channel, and causes the sand to be deposited within shoals where it can be dredged and bypassed to nourish the downdrift beach. The method of dredging the shoals would normally depend on the type of equipment available to the contractors bidding the work. With coordination, maintenance dredging of the channel entrance and shoals could be accomplished in the same contract. Probably the most convenient and efficient method for dredging the shoals off the spurs would be through use of a suction head and pipeline dredge, whereby the material could be deposited in the nearshore on the downdrift side. If necessary, a suction head and sand fluidizer could be combined to allow the dredge to stand off a greater distance from the spurs, jetties, and channel approach. In principle, in mild waves a small cutter-head dredge could also work at the shoals. In the end, however, the type of equipment available and ingenuity of the contractor will determine how the shoals will be dredged, if dredging of the shoals proves to be necessary.

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[11] The first author of this report participated in the field data collection for Sasaki and Sakuramoto (1984) and similar harbor and sedimentation studies in Japan.

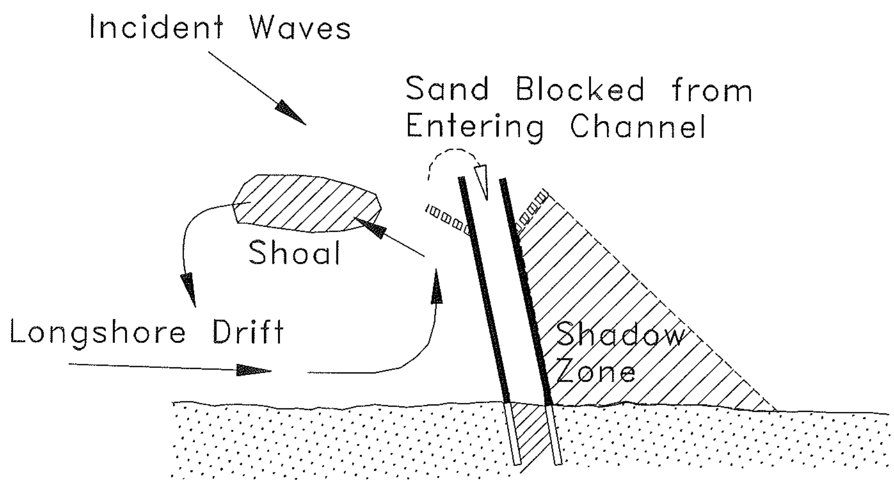

Figure 29. Conceptual functioning of spur jetties.

Because the spurs increase the effective width of the jetties, they also serve to increase the size of the protected area in the lee of the downdrift jetty, known as the jetty "shadow zone," and thus prevent potential erosion along the beach adjacent to the downdrift jetty. During seasonal longshore sediment transport reversals the dual sand interception and shadowing process is reversed, and some of the sand which has accumulated in the updrift shoal may naturally return to the beach. Spurs are expected to provide substantial stability to the directly adjacent north and south beaches. Addition of the spurs to the jetties should not preclude construction of the parallel seaward sections of the jetties between the 10- and 11-ft MSL depth contours, because this extension is necessary to maintain ebb current velocities at the Gulf mouth and to "jet" sediments away from the channel entrance.

The attachment of spurs is recommended as an alternative to increasing jetty length after consideration of the following: (1) the spurs will increase wave diffraction and sheltering and, therefore, reduce the longshore sand transport rate and resultant recession along adjacent shorelines; (2) spurs will require a smaller volume of construction material because they will be in shallower water; (3) spurs are an innovation that was originally developed in response to poor performance of traditional jetty designs; and (4) the potential for the continuous growth of shoreline fillets along the jetties, whereby the effective blocking length of the jetties is gradually reduced (as has occurred updrift at Mansfield Pass), is minimized by the action of the spurs to redirect and force currents away from the channel. At Packery Channel, the spurs should be located just beyond the outer bar crest at the 10-ft depth contour (as shown in Figures 30 and 31) because little sand transport occurs seaward of this location. The length of the spurs should be on the order of 300 ft as tested through shoreline change modeling described later in this chapter.

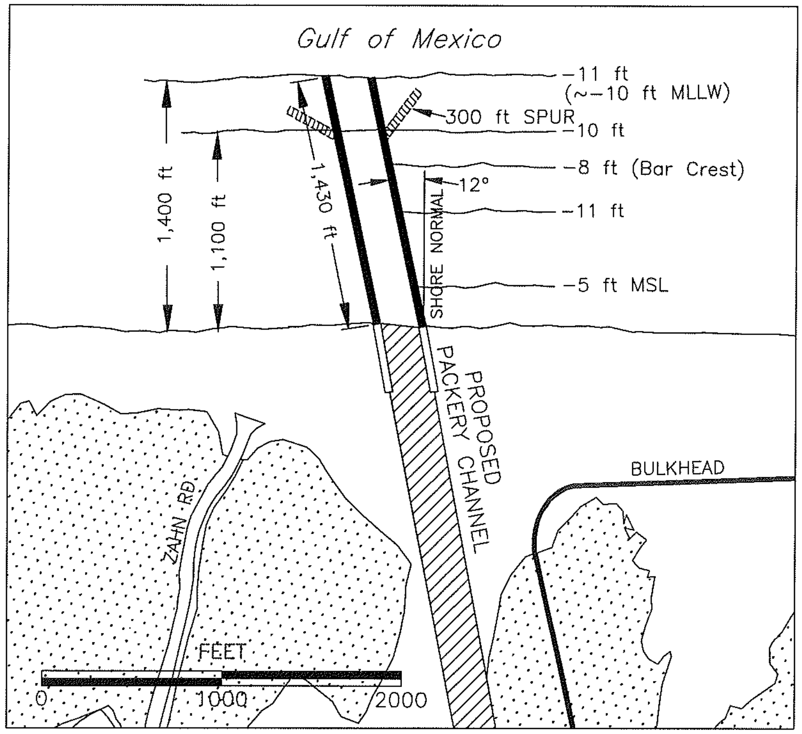

Figure 30. Spur jetty design at Packery Channel.

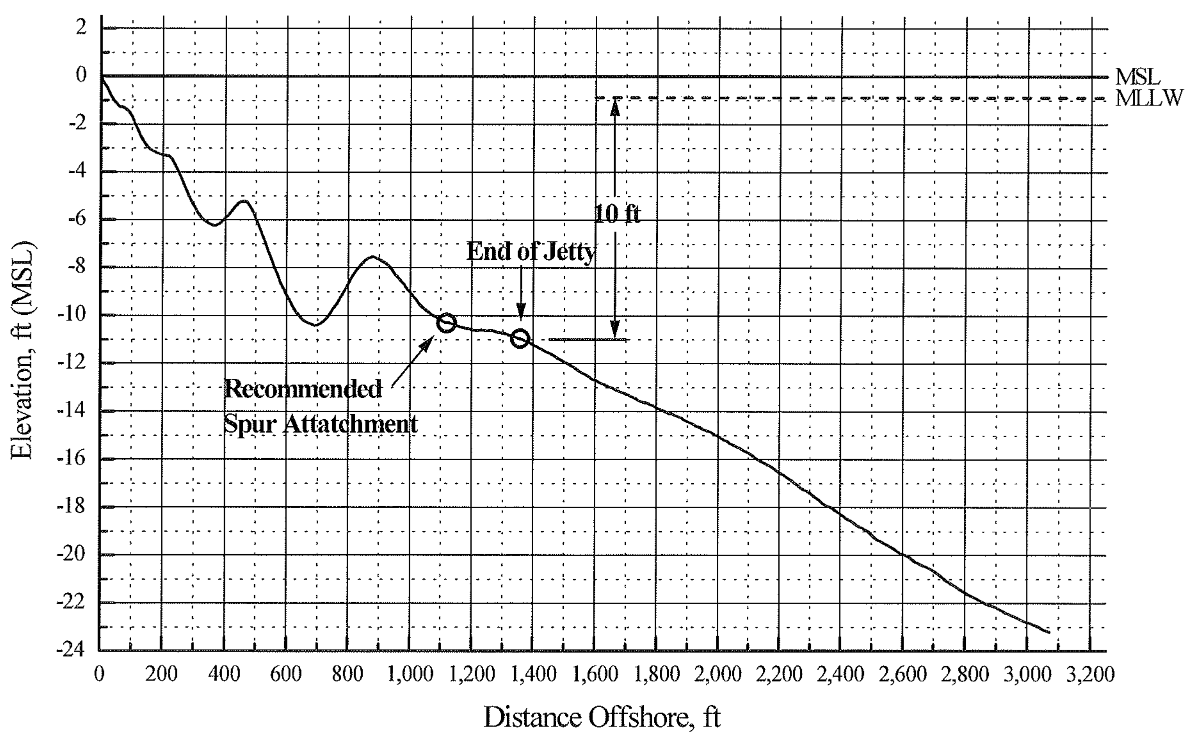

Figure 31. Location of recommended spur attachments along jetties (as shown along calculated average beach profile).

Shoreline Change Modeling

The U.S. Army Corps of Engineers Waterways Experiment Station has developed a shoreline change numerical model as a generalization of mathematical shoreline change models. The model, known as GENESIS (Hanson and Kraus 1989), enables prediction of shoreline evolution under a wide range of beach, coastal structure, wave, and initial and boundary conditions. It can also account for sediment sources (such as beach fills and river discharges), as well as sediment sinks (such as inlets and sand mining). GENESIS modeling was applied to the study in two phases: (1) For calculation of longshore sand transport variability (specifically for use in the sediment budget) and for model calibration; and (2) for prediction of project impacts on evolution of the adjacent shoreline.

Model Calibration

The first phase of GENESIS modeling involved simulation of shoreline change along a 17-mile stretch of beach approximately centered at Packery Channel. Shoreline positions were interpolated along a 100- and 200-ft variably-spaced grid and were based on digitized June, 1974, shoreline position data provided by UT-BEG. The grid was more finely spaced near the Mustang Island Fish Pass, and minor discontinuities along the shoreline (as would exist during the periodic re-opening of natural passes along the modeled reach) were smoothed and connected to the adjacent shoreline positions. Background or historically occurring shoreline recession was included to represent loss of sand due to cross-shore wind transport and storm washover. Recession rates were based on data provided for the period June, 1974, to June, 1982, as given in Paine and Morton (1989) and through comparison of shoreline positions for November, 1968, and January, 1996, which were digitized from aerial photographs. The 1970-1979 wave hindcast data (discussed in Chapter 2) were input to the model to provide continuous wave conditions. As mentioned in Chapter 2, the hindcast data for the years 1972 and 1974 were judged inaccurate and were omitted from the input wave file. The jetties were modeled as a single groin extending 800 ft seaward from the 1974 shoreline position.

GENESIS calculates shoreline response to imposed boundary conditions, engineering structures, and other coastal engineering activities. Note that no post-1974 shoreline data were available for model calibration. The only documented response available in the project reach is that at the Mustang Island Fish Pass. Therefore, the model was considered calibrated when local sediment impoundment could be reproduced updrift and downdrift of the Fish Pass jetties during the 1974 to 1996 simulation period. This impoundment was verified through inspection of the aerial photography in Figure 19 in Chapter 2. The model assumption that the offshore contours are parallel to the shoreline was also verified through aerial photography, as shown in Figure 32. Comparison of January 12, 1972, and January 12, 1996, shoreline maps (digitized from aerial photographs) of the Fish Pass indicated that the beach was stable to accretional within about

3,800 ft updrift and 3,000 ft downdrift of the pass (Figure 33). As shown in Figure 34, model over-prediction of shoreline recession occurred downdrift of the Fish Pass, which suggested that use of GENESIS as a predictive tool at the proposed Packery Channel jetties would result in calculation of downdrift shoreline recession even for short jetties. Such over-prediction was deemed acceptable for conservative (or "worst case") calculation of downdrift beach erosion at Packery Channel.

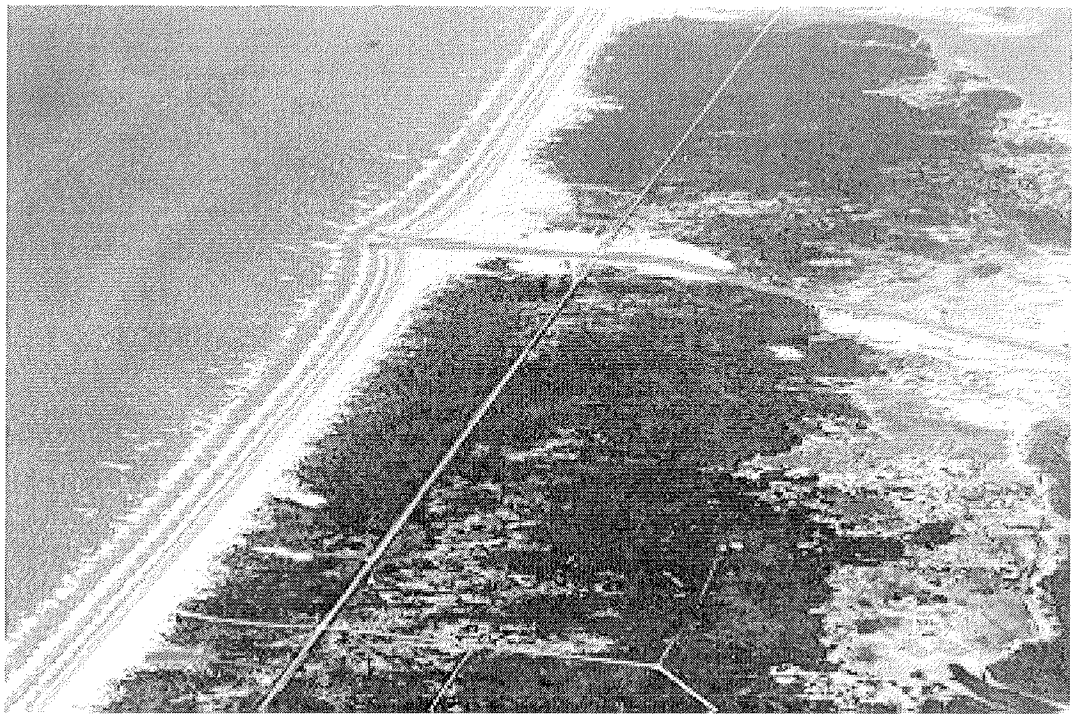

Figure 32. Breaker lines parallel to the shoreline at the Mustang Island Fish Pass, April 6, 1974.

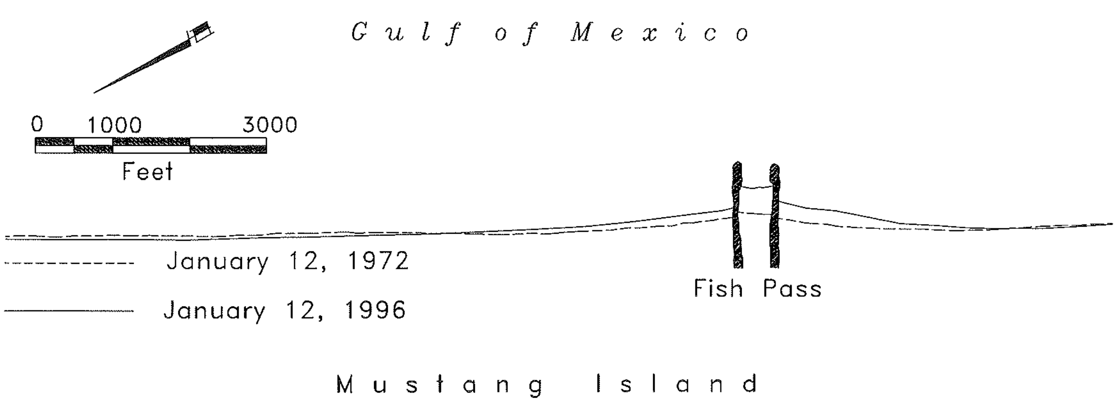

Figure 33. Measured shoreline change at the Fish Pass.

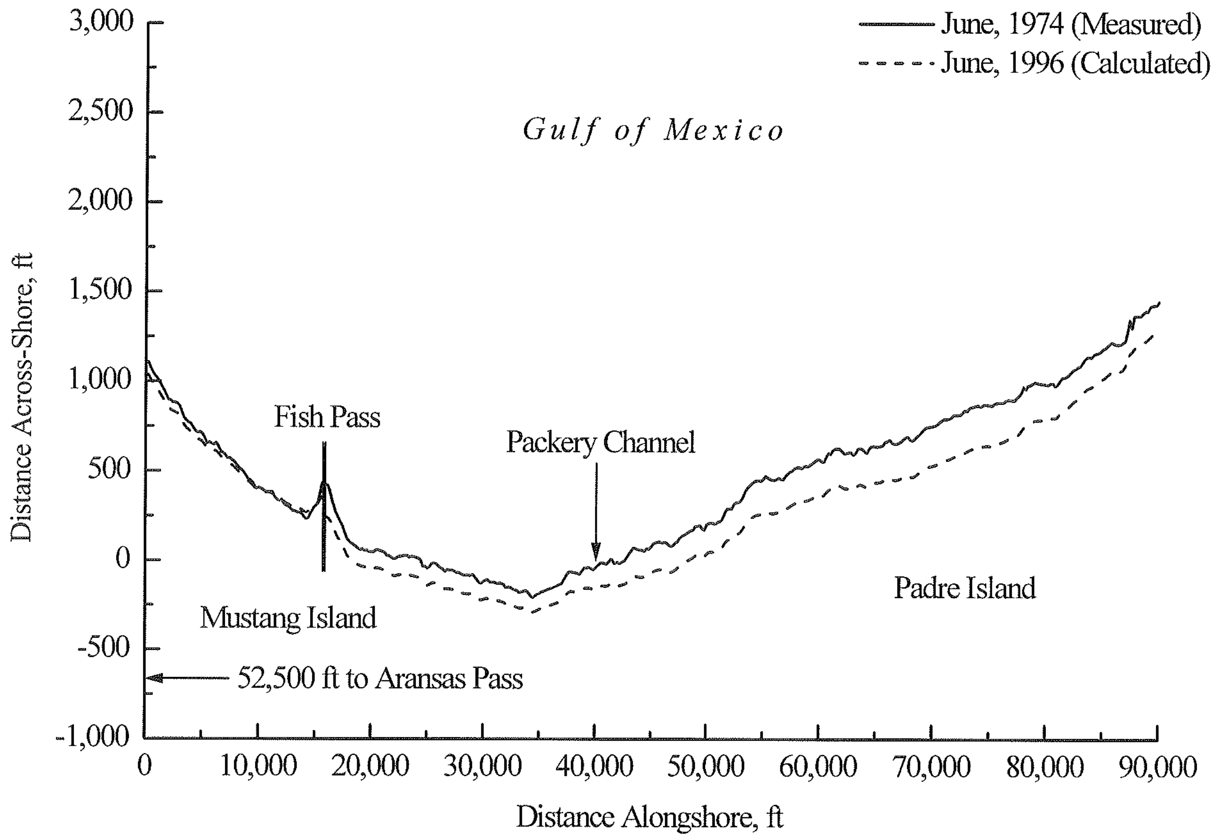

Figure 34. Calibration of GENESIS based on shoreline trends at Fish Pass.

The second phase of GENESIS modeling was conducted to predict project impacts on the shorelines adjacent to the proposed jetties and included a 6.2-mile stretch of beach approximately centered at Packery Channel. The OCTI hindcast wave data were again input as wave conditions. Shoreline positions were interpolated along a 100- to 200-ft variably-spaced grid, with finer grid spacing used near Packery Channel. The 1996 shoreline position shown in Figure 35 was projected from the digitized 1974 shoreline based on published 1974 to 1982 shoreline change trends (Paine and Morton 1989) and was quantitatively verified through comparison of digitized November, 1968, and January, 1996, aerial photographs of the Packery Channel area.

Locations of the northern and southern corners of the north Padre Island seawall were digitized from the January, 1996, aerial photograph so that the structure could be accurately represented in the model grid. Because the seawall is presently threatened by a receding shoreline and will be located on the long-term net downdrift side of the jetties, it was targeted as a critical project-impact area. A central objective of the modeling was to determine if the sediment impoundment at the jetties will result in a change (increase or decrease) in the on-going shoreline recession and if effective sand management can be used to reduce the future risk of beach erosion and seawall undermining.

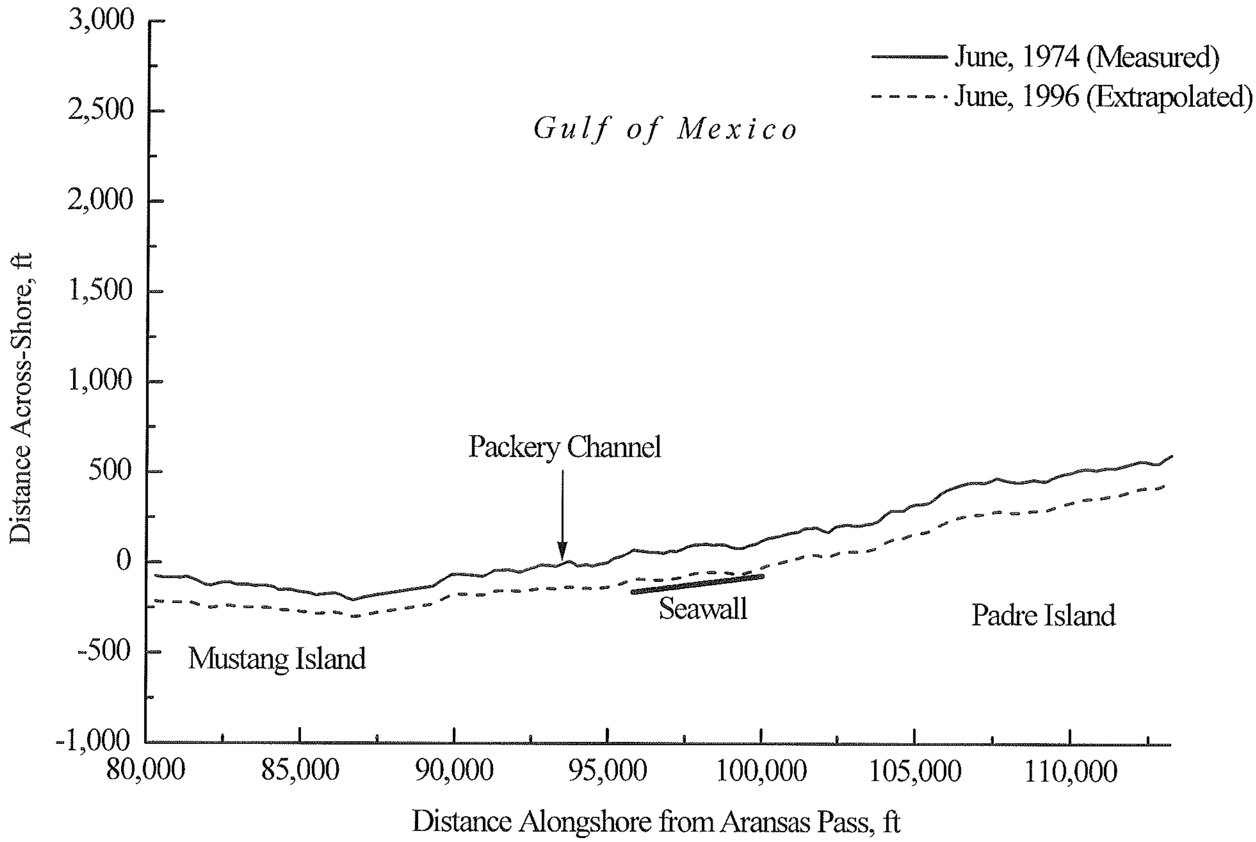

Figure 35. Measured 1974 and empirically extrapolated 1996 shoreline positions near Packery Channel.

Alternatives

After numerous model sensitivity tests and trial runs, project alternatives were developed to predict jetty impacts on adjacent shorelines. These alternatives, as shown in Table 4, included variations in structural configuration, sand management strategies, and transport rate, and they were specifically modeled to maximize shoreline stability downdrift of the proposed jetties and at the seawall and to minimize required project maintenance. Table 5 presents a summary of the project alternatives as applied to specific modeling cases. (Note that in this report, a "model case" is defined as a particular modeling execution and is not used synonymously with "project alternative." That is, several model cases may have been run to test a project alternative.) Jetty length was not varied during the modeling in order to maintain the objective of minimizing project cost (i.e., the minimum required jetty length of 1,400 ft, as determined by inspection of beach morphology, was tested). The primary functional objective for jetty length was to ensure that the jetties traversed the outer bar.

Table 4. Parameters that were or were not varied in the shoreline change model simulations of alternative designs.					
Structure		**Sand Management**		**Transport Rate**	
Jetty Length	No	New-Work Dredged Material Placement	Yes	Longshore Net	Yes
Spurs	Yes	Bypassing: *location* *volume* *cycle*	 Yes Yes Yes	Longshore Gross	Yes
Jetty Angle	No	Wind-Blown Sand	Yes	Background Erosion	Yes

Table 5. Model cases and variables applied.

Variable	CASE 1	CASE 2	CASE 3	CASE 4	CASE 5	CASE 6	CASE 7	CASE 8	CASE 9
Ending Year	2007	2007	2007	2007	1999	2007	2007	2007	2007
Mechanical Bypassing Schedule	----	45,000 cy 2 yr cycle	45,000 cy 2 yr cycle	45,000 cy; 2 yr cycle	45,000 cy 2 yr cycle	128,000 cy 2 yr cycle	45,000 cy 2 yr cycle as needed	45,000 cy 2 yr cycle as needed	45,000 cy 2 yr cycle as needed
Mechanical Bypassing Start Date	----	1998, 1999, 2000	1998	1998	1998	1998	1998	1998	1998
Mechanical Bypassing Discharge Location	----	2000 ft South of Channel	2000 ft South of Channel	4000 ft South of Channel	4000 ft South of Channel	4000 ft South of Channel	4000 ft South of Channel	4000 ft South of Channel	4000 ft South of Channel
Beach Fill Location	----	From Jetty to South end of Seawall	From Jetty to South end of Seawall	From Jetty to South end of Seawall	Only Along Seawall	From Jetty to South end of Seawall	From Jetty to South end of Seawall	From Jetty to South end of Seawall	From Jetty to South end of Seawall
Spur Jetty	----	300 ft, 600 ft	300 ft	300 ft	300 ft	300 ft	300 ft	300 ft	300 ft
Model *K* coefficient	0.55	0.55	0.55	0.55	0.55	0.77	0.55	0.55	0.55
Modify Input Wave Conditions	----	----	----	----	----	----	Add Years of Northward Transport	Rotate Incident Wave Angles	----
Background Shoreline Recession	Based on 1974-1982 Rate	Based on 1974-1982 Rate	Based on 1974-1982 Rate	Based on 1974-1982 Rate	Based on 1974-1982 Rate	Based on 1974-1982 Rate	Based on 1974-1982 Rate	Based on 1974-1982 Rate	50% of 1974-1982 Rate

Alternative 1: No Sand Management

The first alternative tested involved two 1,400-ft long jetties (without spurs) spaced 300 ft apart and extending seaward to the 11-ft MSL depth contour. This alternative had no sand management (no strategy was implemented to reduce downdrift erosion) and was included to emphasize the impact that a project without sand management could have on beach erosion. Background shoreline recession was included using the same rates as during model calibration, and the dimensionless proportionality coefficient K (which relates sand transport rate to the longshore component of wave energy flux) was set to 0.55.

As plotted in Figure 36, jetties at Packery Channel will effectively trap sediment that is transported alongshore by waves. The result of this trapping is seen in the large shoreline fillet that formed updrift of the jetties and the accompanying shoreline recession that occurred downdrift (particularly, along the seawall). Note that this type of shoreline offset has not occurred at the Fish Pass; the jetties at the Fish Pass are apparently too short to greatly restrict the longshore transport, and sand is easily transported around their ends. At Packery channel, the simulated blockage of sediment emphasizes the need for effective sand management.

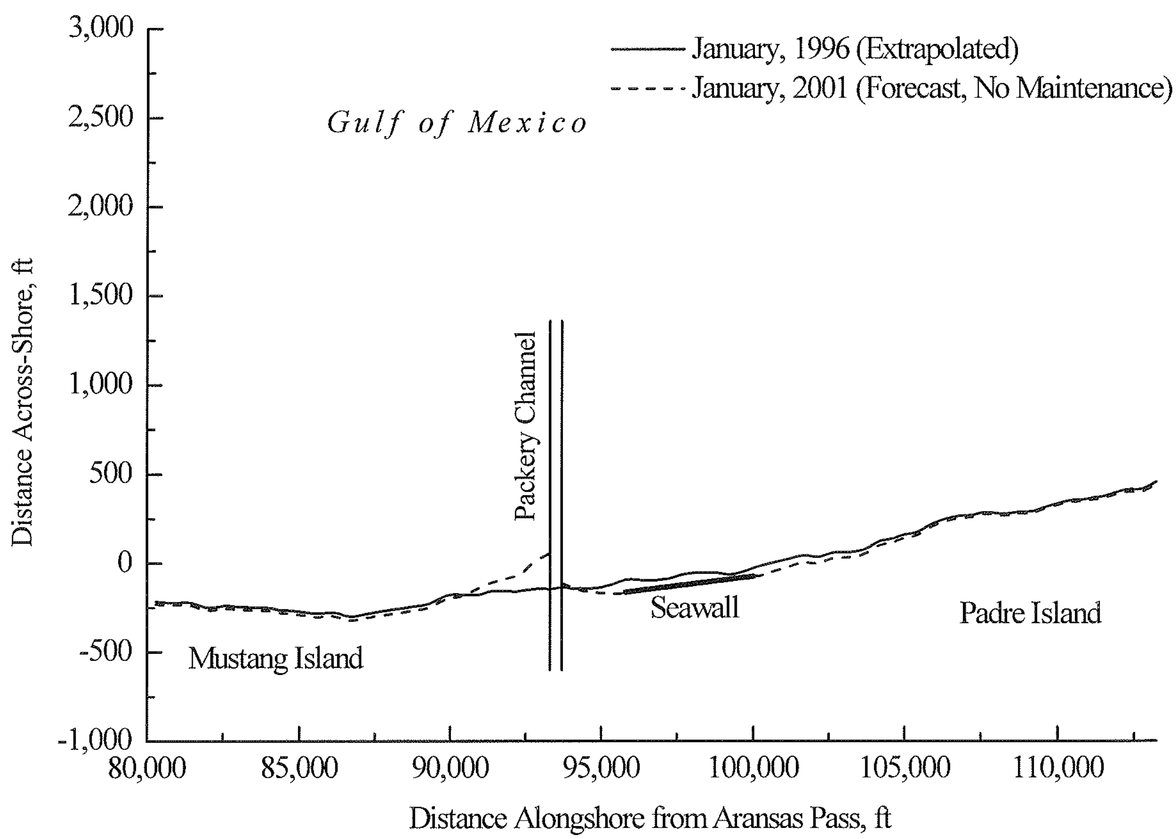

Figure 36. Case 1: Predicted impact on adjacent shorelines if no sand management is initiated.

Alternative 2: Apply Sand Bypassing and Beach Fill

The next project alternative tested included placement of 500,000 cu yd of beach-quality sand, estimated to be available after new-work dredging of the channel, for a beach fill. The fill was placed along 6,600 ft of beach adjacent to the southwest jetty and was represented in GENESIS as an 80-ft addition to the width of the berm (after adjustment to an equilibrium configuration). Mechanical bypassing was modeled as a maintenance feature to prevent excessive fillet formation and associated natural bypassing of littoral sediment around the ends of the jetties and into the channel. The volume of sand that was mechanically bypassed was calculated based on a net longshore transport rate (in the vicinity of the channel for $K = 0.55$) of 45,000 cu yd/yr to the southwest. The bypassed sediment was placed on the downdrift beach approximately 2,000 ft south of the channel (near the north end of the seawall).

Several model runs were conducted to determine that mechanical bypassing should be performed 2 years after jetty construction and continued at 2-year intervals to avoid excessive entrance channel shoaling. Background shoreline recession was again modeled based on the 1974-1982 measured rates published by UT-BEG and the digitized 1968 and 1996 aerial photographs. As shown in Figure 37, construction and effective maintenance of the channel reduces the shoreline recession between the seawall and the channel.

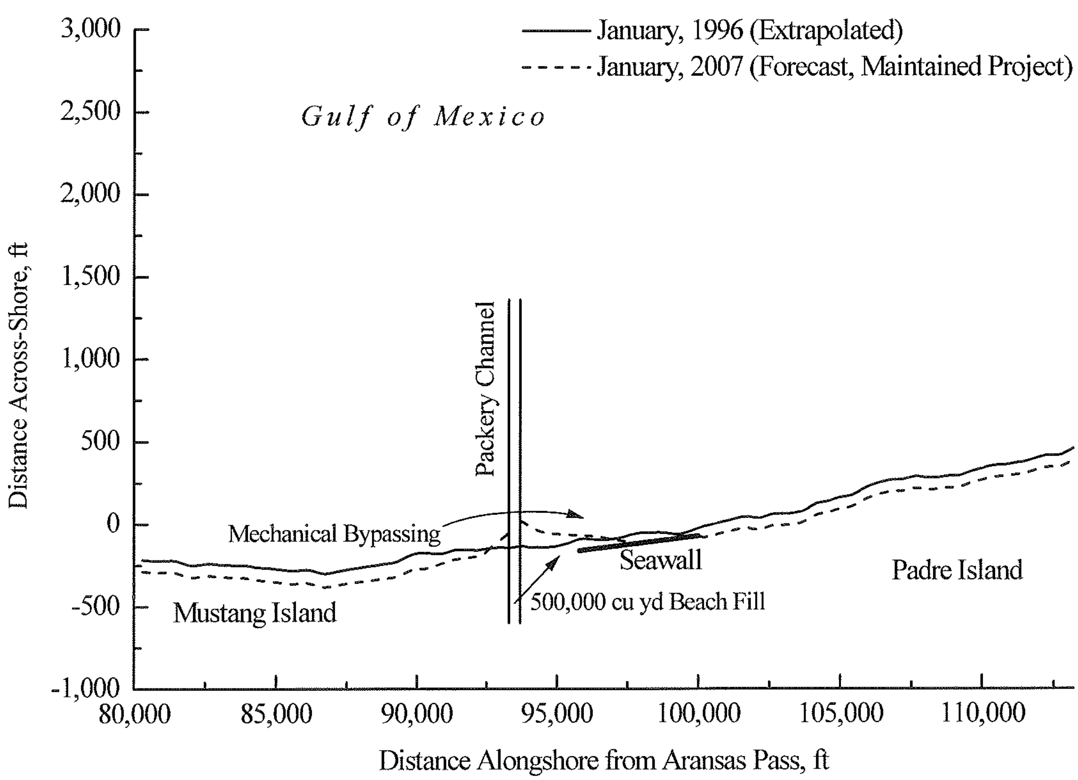

Figure 37. Case 2: Maintained project (forecast through year 2007).

Alternative 3: Spur Jetties

After verifying the benefit of including sand management (particularly, sand bypassing and beach fill) within project alternatives, several jetty configurations were modeled including construction of spurs to minimize the flow of sand around the jetties and to confine sediment impoundment within an area between the jetties, spurs, and natural beach. By controlling the location of sediment impoundment, the sand is conveniently "stockpiled" between maintenance dredging (mechanical bypassing) operations. Both 300-ft and 600-ft long spurs were added to the conditions used in Case 2 (Figure 37). Because the spur-jetty modeling indicated that no significant advantage was gained with longer spurs, a 300-ft long segment was judged to be adequate for the spur length. Although not apparent in Figure 38 due to the plot scale, a quantitative comparison of performance of the spur and non-spur jetty configurations revealed that the spurs increase the wave shadowing area of the jetties and reduce the transport of sand around the ends of the jetties by about 20%.

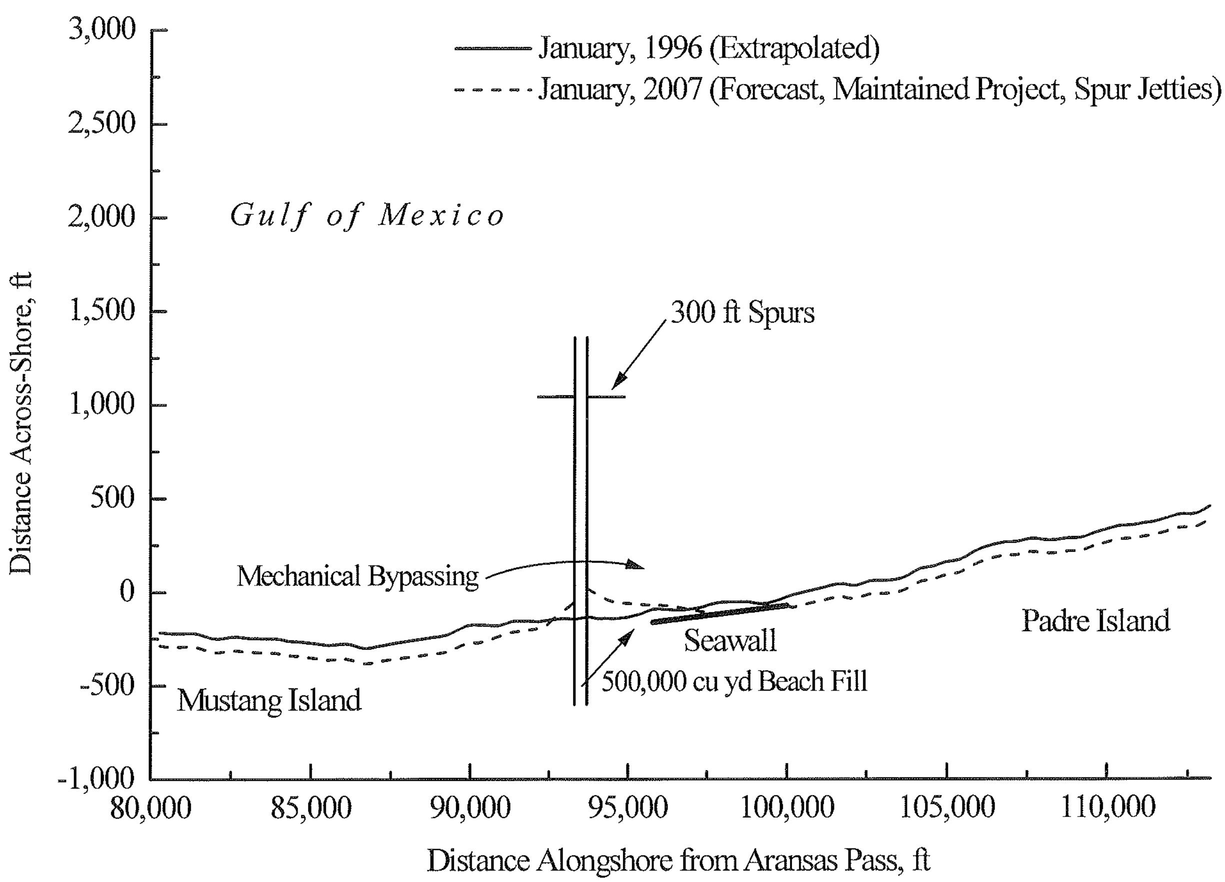

Figure 38. Case 3: Use of 300 ft spurs to contain sand for mechanical bypassing.

Alternative 4: Vary Location of Bypassed Sand Discharge

As a possible means of further optimizing project sand management, an alternative was modeled to test the potential benefit of discharging the mechanically-bypassed sediment at the center of the seawall instead of at the north end of the seawall (4,000 ft downdrift of the channel instead of 2,000 ft downdrift of the channel). The simulation results shown in Figure 39 were calculated under the same conditions applied for Alternative 3 as shown in Figure 38, with the exception that the bypassed-sediment discharge zone was at the center of the seawall instead of at the north end of the seawall. Note that although the offshore excursion of the downdrift shoreline fillet is decreased, the alongshore excursion is increased, thereby reducing the risks associated with shoreline recession. The mechanical sand bypassing conditions of Alternative 4 are considered ideal for maximization of potential benefits to the seawall.

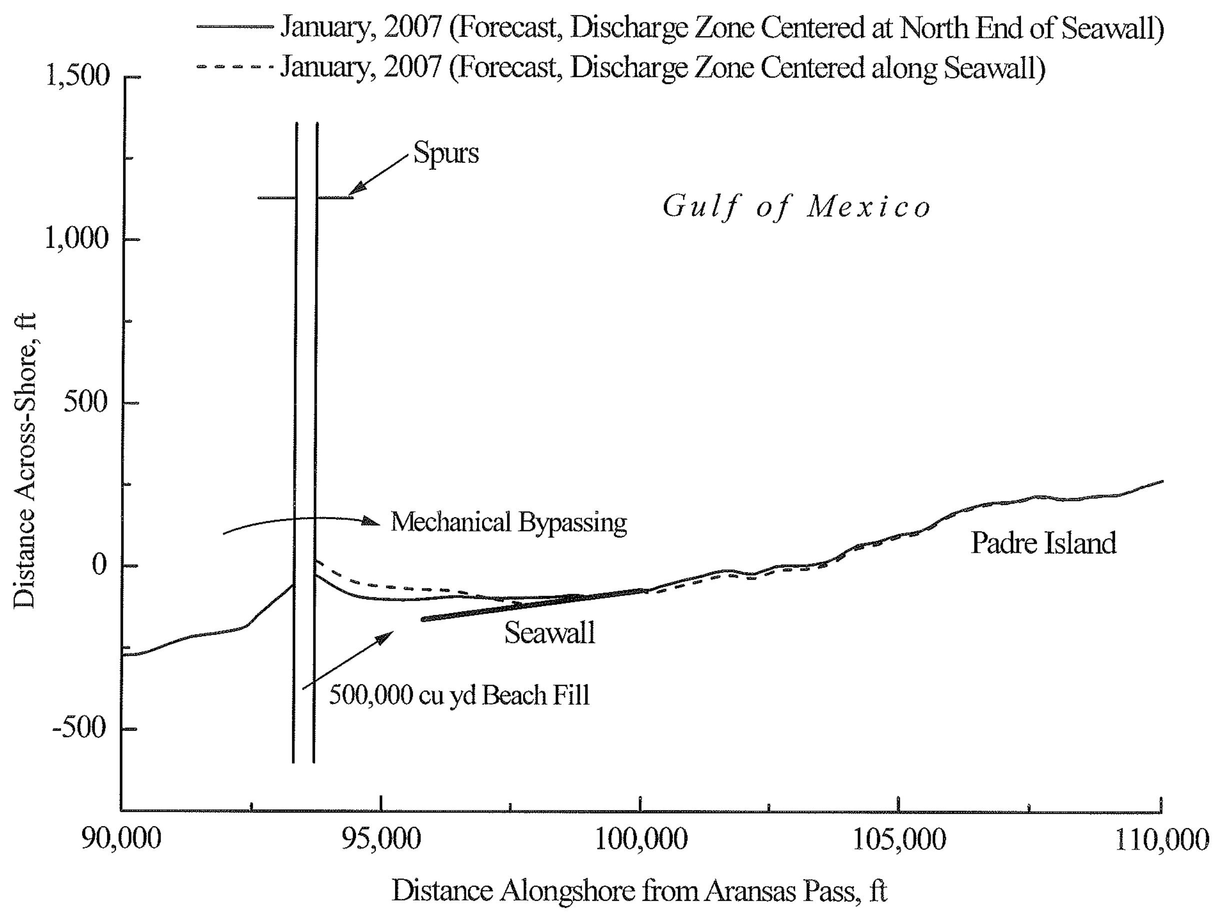

Figure 39. Case 4: Vary location of mechanical bypassing sand discharge zone.

Alternative 5: Vary Location of Beach Fill

Investigation of sand management alternatives was continued by conducting a simulation to test the sensitivity of the downdrift shoreline to the configuration and location of the initial (new-work dredged material) 500,000 cu yd beach fill. Previously discussed alternatives had included a 6,600-ft long, 80-ft wide fill extending from the south jetty to the south end of the seawall. Figure 40 plots the results of simultaneously decreasing the fill length to approximately 4,000 ft, increasing the fill width to approximately 140 ft, and centering the fill along the seawall. Note that, compared to the fill as previously configured and located, this case resulted in moderate short-term (3-year) variation in shoreline positions downdrift of the channel; however, the longer-term (11-year) variation was insignificant and therefore not shown.

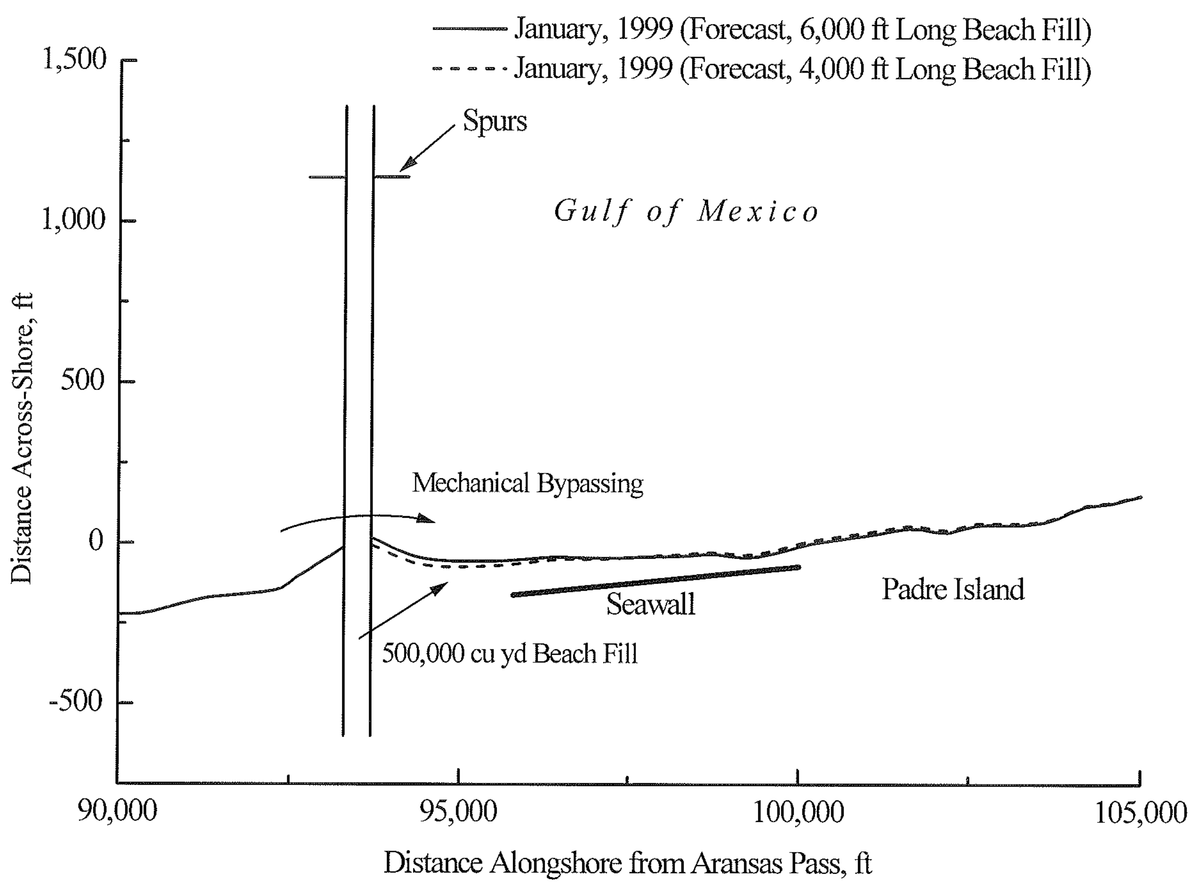

Figure 40. Case 5: Vary configuration and location of beach fill.

Summary of Project Alternatives

Various sand-management alternatives were investigated based on a local sediment budget and numerical modeling to minimize adjacent shoreline impacts of the project. Both a beach fill (using approximately 500,000 cu yd of beach-quality sand expected to be available after new-work dredging of the channel) and mechanical sand bypassing were determined to be critical factors of the sand-management plan.

The recommended beach fill placement is south of the channel between the south jetty and the south end of the seawall (see Alternative 4). This sand could also be placed entirely along the seawall to satisfy immediate needs for a wide beach along the structure; however, the long-term (3-to-5 year) added benefit of placing the entire volume of sand only along the seawall is minimal because the fill will spread to adjacent beaches within a few years by wave action. Placement of some of the sand adjacent to the south jetty will protect the jetty from being flanked if severe storms occur during the first few years following project completion, thereby giving time for a critical area of impounded sand to develop within the jetty "shadow zone."

The mechanical bypassing should transfer approximately 90,000 cu yd of sand from north to south of the channel in an approximate 2-year cycle starting two years after jetty construction. The recommended bypassed-sand discharge location is along the center of the seawall approximately 4,000 ft south of the inlet entrance. Placement of the bypassed sand nearer to the channel would probably reduce bypassing operation costs, but would also result in a delay in benefit to downdrift beaches because the jetties will block the northeast waves necessary to transport the sand alongshore (i.e., the sand would be trapped by the jetties).

Nearshore mound placement (McLellan 1990, McLellan and Kraus 1991) was also considered as a possible means of providing dredged material to the beaches south of the jetties. However, for several reasons dredged mound creation was rejected in favor of direct placement on the beach. First, to provide the most benefit, a mound would need to be created directly offshore of the downdrift beach, where the sand could potentially be transported back into the channel during periods of northward longshore transport. Second, construction of the mound might place requirements on dredging equipment and thereby decrease competition (increase cost) in the bidding process. Third, a site-designation process would have to occur prior to placement, probably delaying the operation and adding cost. Finally, the degree of success and time frame for realizing a benefit of nearshore mound placement are unknown. The material may or may not move from the mound and towards the shore within the project time frame, and only a certain percentage will actually reach the beach. In contrast, by placing all material on the beach as in a beach fill, the full benefit of the material for both protecting the shore against long-term erosion and catastrophic storm-induced erosion and creating a wider recreational beach are obtained.

Testing of project alternatives provided insight to the various strategies that should be applied for reduction of downdrift beach erosion and effective sand management. However, these strategies should not be considered as solutions to the problem of seawall-undermining that could eventually occur as a result of the long-term trend of shoreline recession along Mustang and north Padre Islands. As plotted in Figure 41a, the recession should be expected to continue regardless of continued sand-bypassing and sediment impoundment updrift and downdrift of the jetties. Note that although the beach will remain stable near the jetties, erosion is predicted downdrift of the seawall, as has occurred downdrift of the Galveston, Texas, seawall. This erosion will occur at the north Padre Island seawall regardless of construction at Packery Channel (Figure 41b). Along both the Galveston Island and north Padre Island seawalls, the shoreline continues to naturally recede, and beach width cannot be maintained in front of the structures without sand management. This exposure of the seawall to wave attack and the associated risk of seawall undermining and collapse emphasizes the requirement for continuous project and beach maintenance after jetty construction.

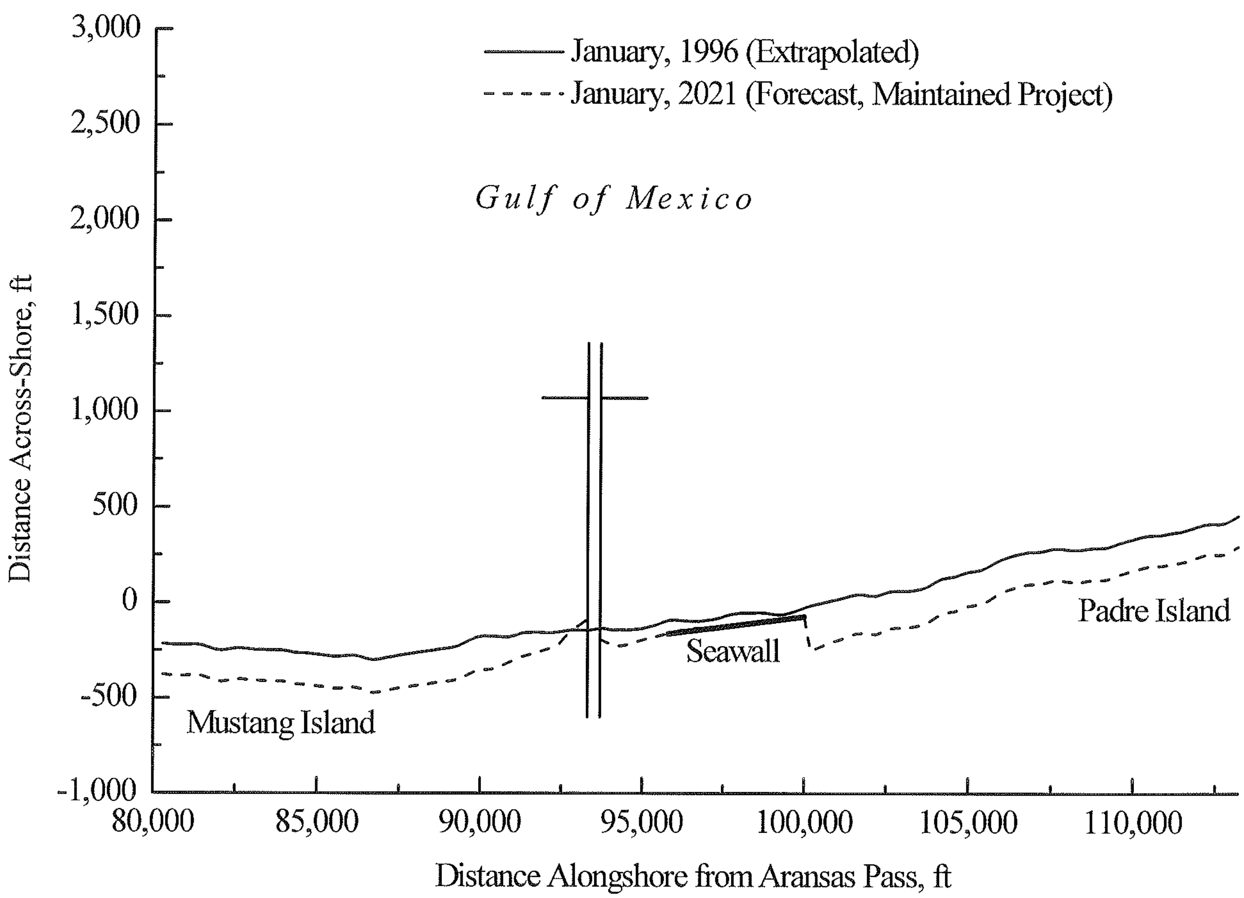

Figure 41a. Forecast of sand impoundment and shoreline recession through year 2021 (with project).

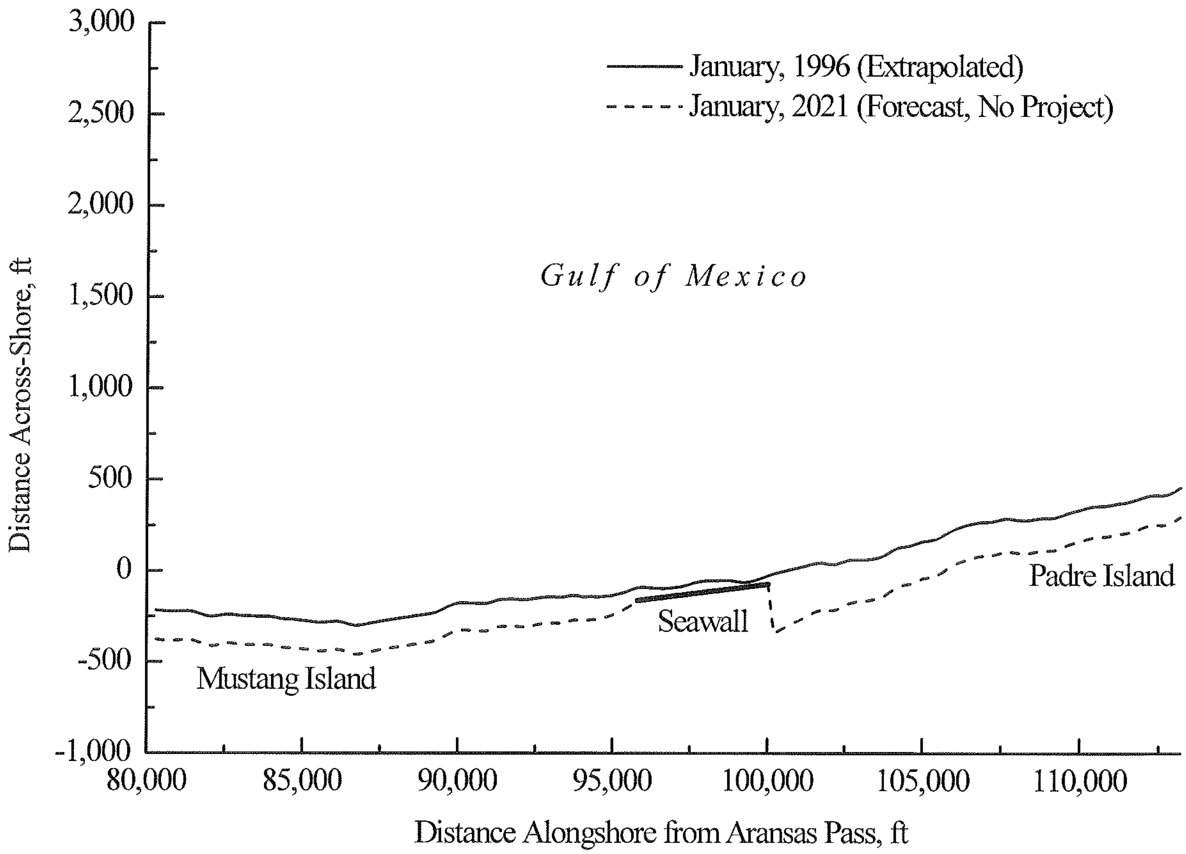

Figure 41b. Forecast of shoreline recession through year 2021 (without project).

Transport Rate Variability

Because it is not possible to predict the exact environmental conditions that will be responsible for future shoreline change, the present study considers a realistic range of variability for forecasting these conditions. Input transport conditions were varied to test an optimal jetty configuration and sand management plan that were selected based on the modeling results discussed up to this point. The direction and magnitude of longshore sand transport and cross-shore transport (wind-blown sand and periodic storm overwash represented through background shoreline recession) were varied based on the ranges of values developed in the local sediment budget and qualitative analysis of historic aerial photography of local beaches.

As described in the previous section, the optimal jetty configuration was determined to include two 300-ft spurs, an entrance channel width of 300 ft, and two parallel jetties extending 1,400 ft seaward to the 11-ft MSL depth contour. The associated sand management plan included an initial 500,000 cu yd beach fill (using the sand from the new-work channel dredging) placed along 6,600 ft of beach from the south jetty to the south end of the seawall, and mechanical bypassing of sand to be discharged near the center of the seawall beginning two years after jetty construction and continuing at a 2-year interval. The volume of sand bypassed in the simulations depended on the net average longshore transport rate, which varied according to the specific situation.

The first variability test case for the selected jetty configuration and sand management plan involved increasing the longshore transport rate near Packery Channel from a net average 45,000 cu yd/yr to 64,000 cu yd/yr by changing the model coefficient K from 0.55 to 0.77. For this case, 128,000 cu yd of sand were mechanically bypassed every other year. As plotted in Figure 42, an increase in net average longshore transport rate produced little change to the predicted updrift and downdrift shoreline position as long as the associated mechanical bypassing volume was increased accordingly. As an upper limit for the project, mechanical bypassing may occasionally transfer a sand volume to the downdrift beach of about 128,000 cu yd every 2 years.

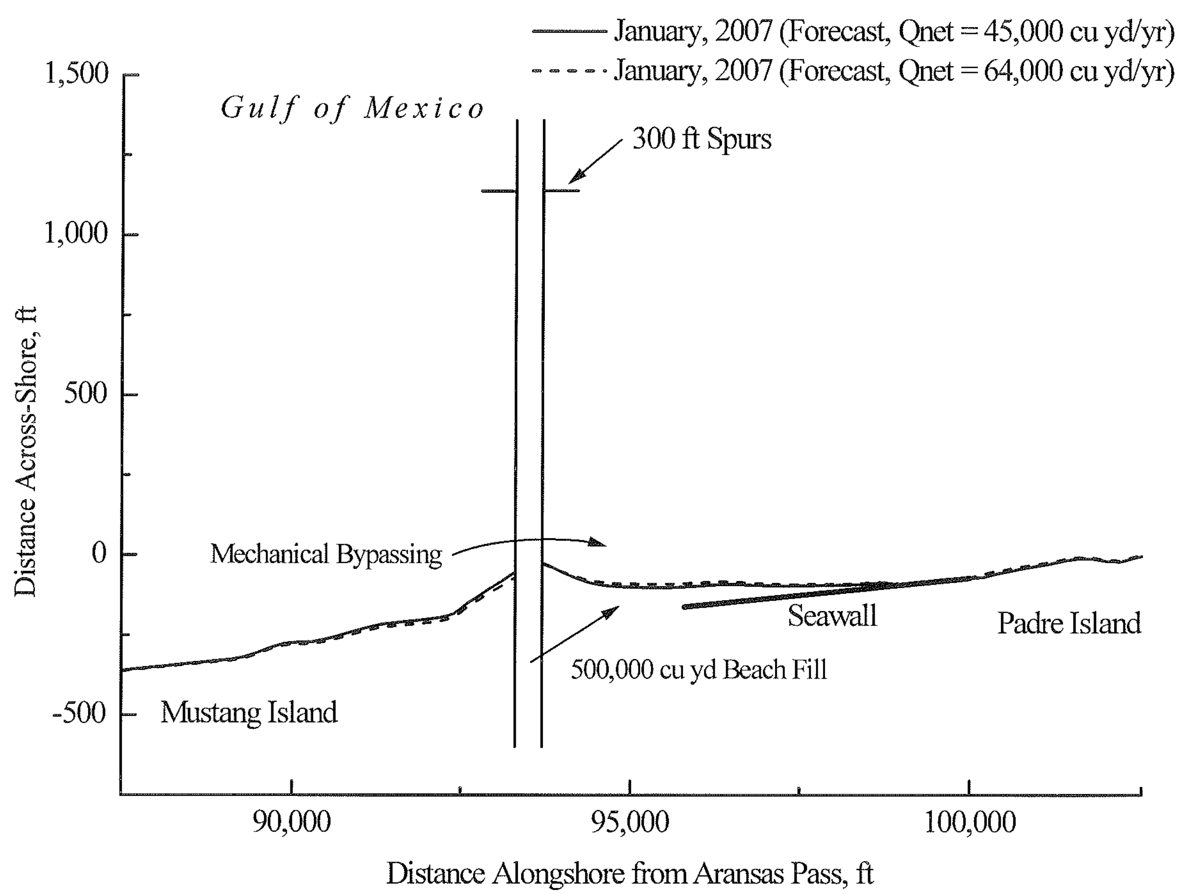

Figure 42. Case 6: Vary longshore transport rate.

Because it is recognized that the direction of net longshore transport may not always be to the south during any given future short-term time span, the next variability test of the selected jetty configuration and sand management plan varied the sequence of incident wave approach. For example, during any given 10-year period, the net average transport may be to the south, whereas during a specific 3-year period within those 10 years the net average transport may be to the north (in the long-term net updrift direction). A simulation was conducted in which the input wave file was modified so that years during which the net transport was to the north were repeated, resulting in net transport to the north during the last 4 years of an 11-year simulation. For this case, no mechanical bypassing was performed during years when the net transport was to the north.

As shown in Figure 43, model results indicate that under the low transport rate (for this case) that occurred when transport was to the north, the associated shoreline fillet at the south jetty did not extend far enough south to protect the entire length of the seawall. Although under this situation mechanical bypassing was not required, a certain volume of sand (on the order of 50,000 cu yd/yr) would be available for placement along the seawall during routine maintenance dredging of the entrance channel.

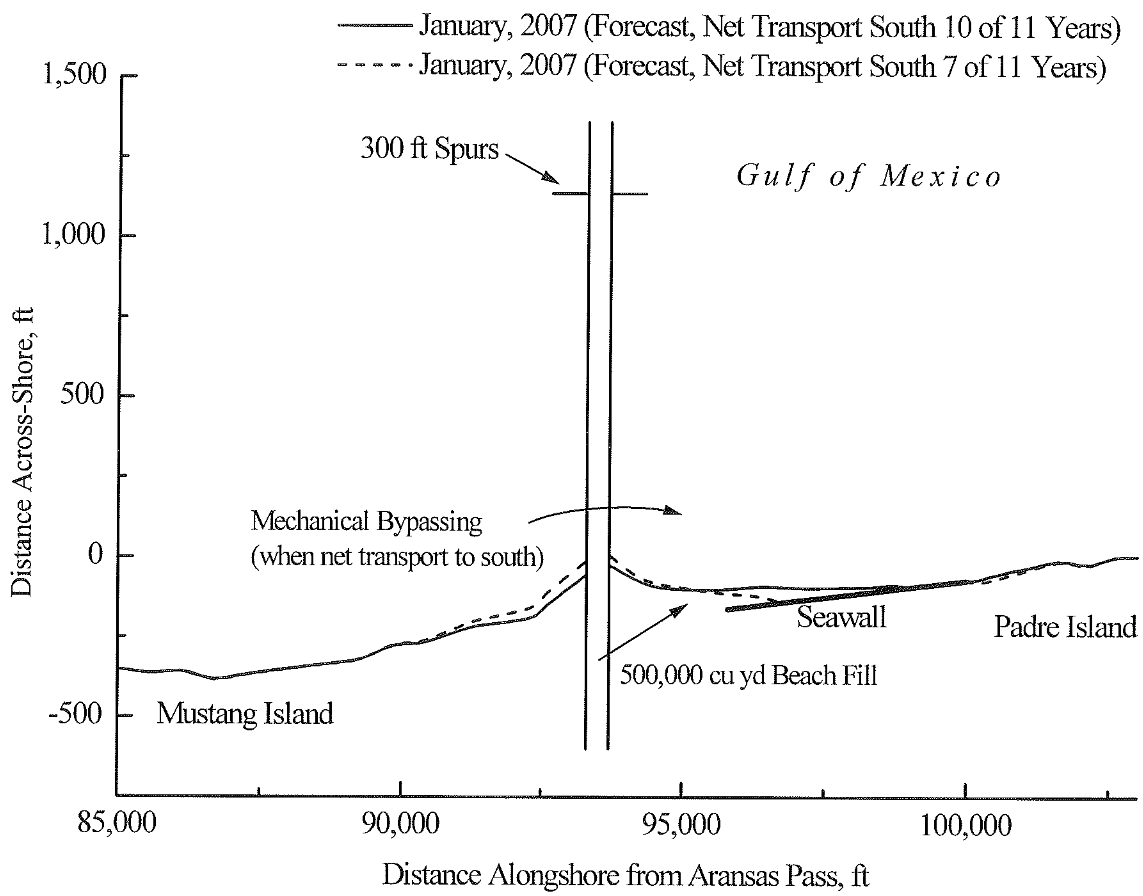

Figure 43. Case 7: Vary sequence of wave conditions in input wave file.

Variability testing of the selected jetty configuration and sand management plan continued through further manipulation of the input wave conditions to predict the shoreline change in response to a sequence of years during which the net transport was northward. Again, the input wave file was modified so that years during which the net transport was to the north were repeated, resulting in net transport to the north during the last four years of an 11-year simulation. For this case, the incident wave orthogonals during the last four years of the simulation were rotated 3, 5, and 10 deg clockwise to increase the average net rate of northward transport by as much as 14,000 cu yd/yr. As plotted in Figure 44, the fillet that formed at the south jetty did not reach far enough alongshore to protect the entire length of the seawall. Similar to the previous variability test (where only wave sequence was modified), a certain volume of sand dredged during maintenance dredging of the entrance channel would be available for placement along the seawall-fronting beach, if necessary.

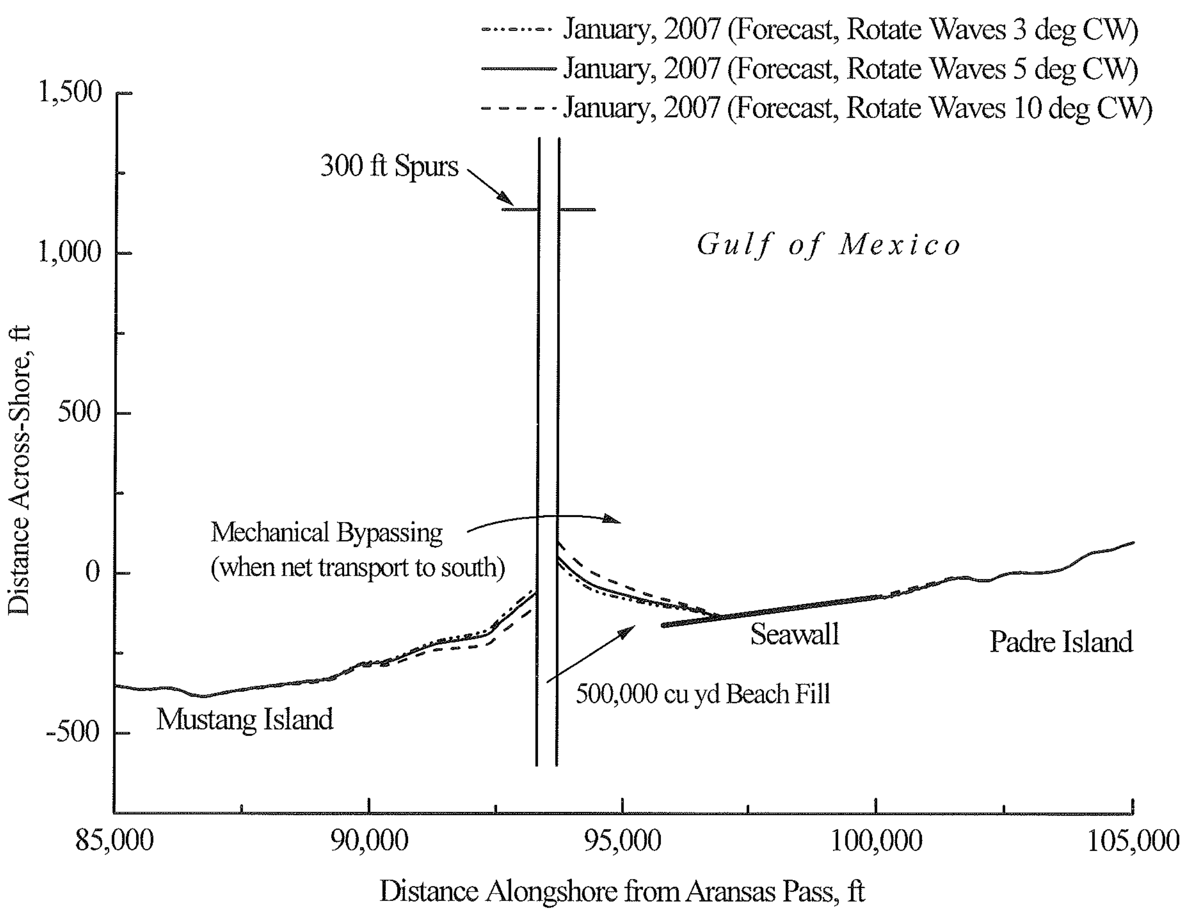

Figure 44. Case 8: Increase angle of incident wave approach.

The final environmental condition that was varied for examination of the selected jetty configuration and sand management plan was the background shoreline recession rate, which was included to account for cross-shore beach losses associated with wind-blown sand transport and storm washover. As previously discussed, the background recession rate was originally based on a projection of published 1974 to 1982 shoreline change rates (Paine and Morton 1989) through the year in which the model simulation ended. Quantitative analysis of 1968 and 1996 aerial photography of the beaches from Newport Pass to the southwest end of the seawall indicated that projected shoreline change rates based on the 1974 and 1982 data are realistic. Because average long-term rates of shoreline recession for the study area are less than the more recent rates of recession[12], a simulation was performed to forecast adjacent shoreline response if the background shoreline recession rates continued at only 50% of the measured 1974-1982 rates. As plotted in Figure 45, a reduction in modeled background recession rates significantly reduced the erosion along the seawall-fronting beach. Note that even in the case of reduced future rates of shoreline recession, sand management would be necessary to minimize downdrift beach erosion and the eventual recession of the shoreline along the seawall.

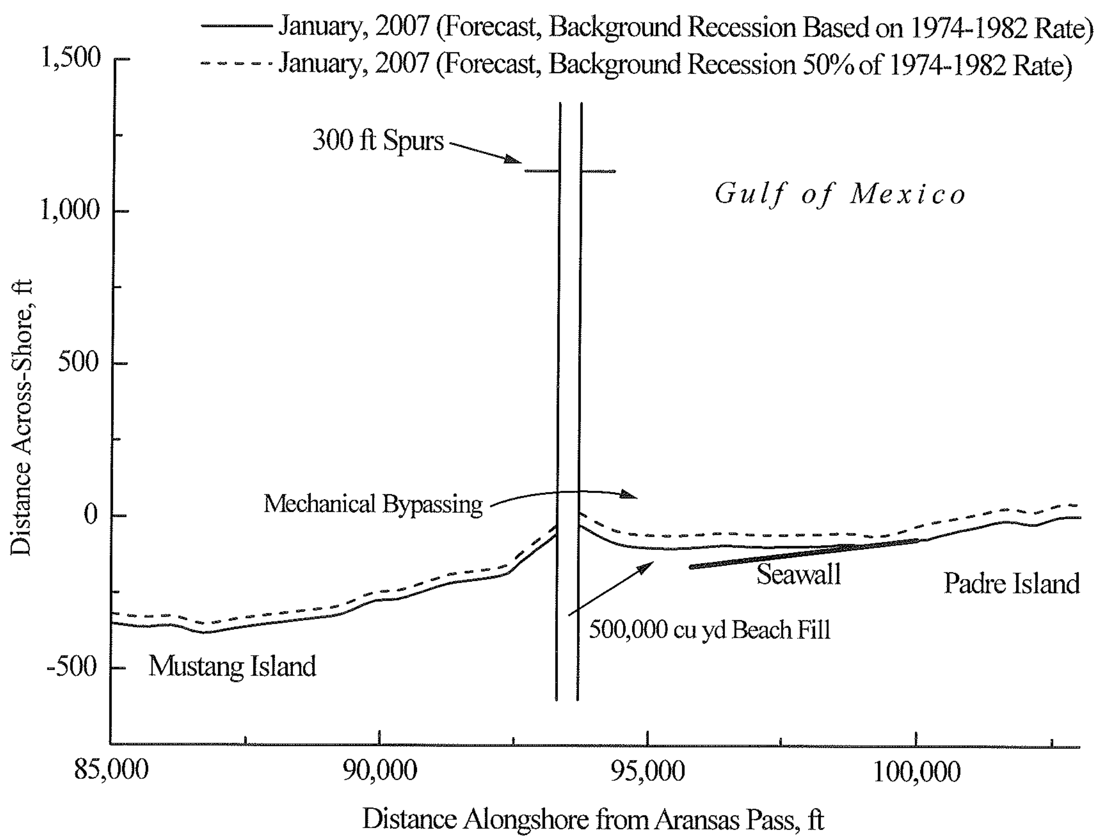

Figure 45. Case 9: Reduce background shoreline recession rates by 50%.

[12] Morton (1993) has suggested that the rates of shoreline recession along the Texas coast are increasing.

4. Inlet Hydrodynamics and Stability

This chapter treats two subjects, inlet stability and navigation safety, both involving the strength of the current. An ebb current tends to transport sediment out of a channel entrance and deposit it in or seaward of the mouth of the channel in the flow divergence zone, where the current magnitude decreases. A strong ebb current poses a danger to navigation through steepening of the incident waves. A flood current tends to transport sediment out of a channel and deposit it in a flood tidal shoal in the bay (for the present situation, in the deposition basin). Both the ebb and flood currents have the potential for eroding the side banks of a channel, potentially requiring construction of bulkheads as a shore-protection measure in areas of strong flow. Therefore, a central element of the analysis presented in this chapter is consideration of the current in the inlet channel for judging the balance between inlet stability and navigation safety. Dredging maintenance requirements and bank erosion are also considered.

Analytic Model of Inlet Hydrodynamics

In this section we review a simple mathematical model as a means of identifying some of the main parameters controlling the strength of the current in an inlet channel. The model also allows direct examination of an anecdotal conclusion that has been made by some parties that the flow in the proposed Packery Channel will be considerably less than that in the Mustang Island Fish Pass, thereby making Packery Channel less stable and more susceptible to closing than the Fish Pass. Results of application of more rigorous numerical models of the hydrodynamics of Packery Channel are described in Report 2 (Brown and Militello 1997).

Following Keulegan (1967), Bruun (1990, 1991), and others, in the absence of wind forcing, freshwater discharges, and non-tidal coastal and bay currents, the one-dimensional (along-channel), depth-averaged momentum equation for the along-channel current speed V in an inlet is given by

$$\eta_G - \eta_B = \frac{L}{g}\frac{\partial V}{\partial t} + \left(k_{en} + k_{ex} + \frac{c_f L}{R}\right)\frac{V|V|}{2g} \tag{10}$$

where η_G and η_B are the water-surface elevation with respect to MSL in the Gulf and bay, respectively, L is the length of the channel, g is the acceleration due to gravity (32.2 ft/sec), k_{en} and k_{ex} are entrance and exit head loss (turbulence loss) coefficients, c_f is the bottom friction coefficient, and R is the hydraulic radius. Notation is defined in Figure 46. In arriving at Eq. (10), the tidal amplitude was assumed to be small compared to the mean water depth in the channel, among other simplifying assumptions. This assumption is reasonable for Packery

Channel, because the mean tidal range is 1.3 ft as compared to the channel design water depth of 11 ft.

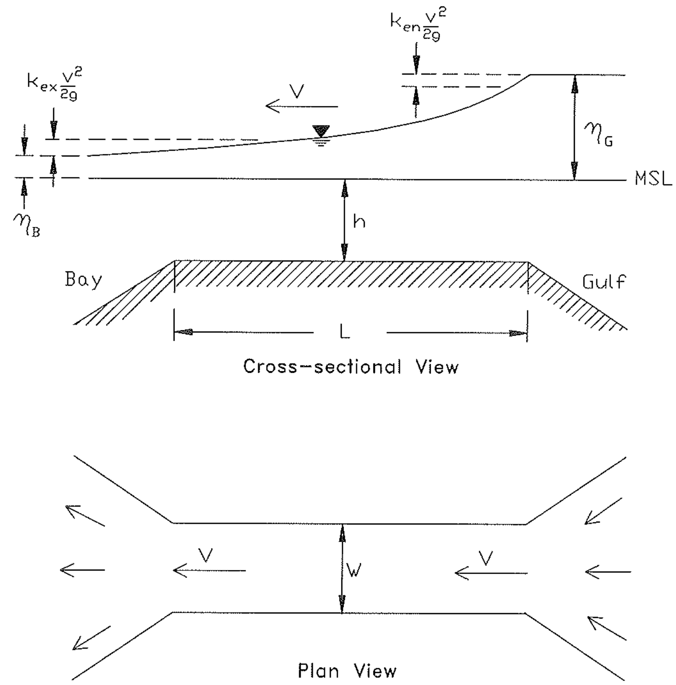

Figure 46. Notation used in momentum equation.

The hydraulic radius is equal to the cross-sectional area of the channel divided by the wetted perimeter. For a rectangular channel with depth h and width W, we have $R = hW/(2h + W)$. If the nominal width is much greater than the nominal depth, as is the case for the proposed Packery Channel (and as was the case for the Mustang Island Fish Pass), then $R \cong h$.

The bottom friction coefficient for gradually varying flow is given by

$$c_f = \frac{8gn^2}{R^{1/3}} \qquad 1.49 \qquad (11)$$

where n is the Manning coefficient. For a sandy bottom, a representative value is $n = 0.025$ (Chow 1959). Watson and Behrens (1976) determined values of n from measurements of water surface elevation slope along different sections of the Mustang Island Fish Pass (see Chapter 2 for discussion of the Fish Pass) and obtained an average value of 0.028 for the entire channel. A representative value of an entrance or exit head loss coefficient is 0.5, and that value will be substituted here.

Because the acceleration of the current is small in typical tidal flow, we can set $\partial V/\partial t = 0$. Then Eq. (10) can be solved for V to give

$$V = \sqrt{\frac{2g(\eta_G - \eta_B)}{1 + c_f \dfrac{L}{R}}} \tag{12}$$

Except for the factor of unity in the denominator, Eq. (12) is equivalent to an empirical formula (Mannings Formula for uniform flow, which does not contain entrance and exit losses) applied by Watson and Behrens (1976) for analyzing values of n. From Eq. (12), to a first approximation, the speed of the tidal current in the channel depends upon the square root of the difference in water surface elevations along the channel, the bottom friction coefficient, and the ratio of the channel length to depth $L/R \cong L/h$.

Eq. (13) allows a comparison to be made of the relative magnitude of the current in the Mustang Island Fish Pass (see Chapter 2) and that in the proposed channel. For the comparison, we assume that the water elevations are the same at the locations of both passes, and also that the length and depth of the Fish Pass are 10,000 ft and 8 ft, respectively, and the length and width of the proposed channel are 15,000 ft and 11 ft. The 8-ft depth of the Fish Pass is the design depth; without maintenance of depth by dredging, the Fish Pass shoaled to a mean depth of 4 to 5 ft (Watson and Behrens 1976). The 11-ft depth for Packery Channel is that expected with maintenance dredging. Then, with the notation "FP" denoting the Fish Pass and "PC" denoting Packery Channel, we have $(L/h)_{FP} = 10{,}000/8 = 1{,}250$, and $(L/h)_{PC} = 15{,}000/11 = 1{,}400$. However, the several curves along the route of the existing portion of Packery Channel will increase the friction coefficient somewhat over that of the straighter alignment of the Fish Pass.

$$\frac{V_{PC}}{V_{FP}} = \sqrt{\frac{1 + c_f\left(\dfrac{L}{h}\right)_{FP}}{1 + c_f\left(\dfrac{L}{h}\right)_{PC}}} \tag{13}$$

For a representative value of $c_f = 0.05$, the products $c_f\,(L/h)$ are much greater than unity and so

$$\frac{V_{PC}}{V_{FP}} \cong \sqrt{\frac{\left(\dfrac{L}{h}\right)_{FP}}{\left(\dfrac{L}{h}\right)_{PC}}} \tag{14}$$

Eq. (14) leads to the estimate $V_{PC}/\,V_{FP} \approx 0.95$. That is, the hydraulic efficiencies of the *proposed* design of Packery Channel and the *original as-built* design of the Fish Pass channel are approximately the same.

The current velocity in the Fish Pass was measured by Behrens, *et al.* (1977) and Watson and Behrens (1976), with a representative value of 1.5 ft/sec. For further discussion, we substitute a depth of 5 ft in the above for estimating the actual, not design, ratio of $(L/h)_{FP} = 10{,}000/5 = 2{,}000$. Then, $V_{PC} / V_{FP} = (2{,}000/1{,}400)^{1/2} = 1.2$. Therefore, by this simple model and not considering details of the hydrodynamics and meteorology, the tidal current in the proposed Packery Channel, if the channel is maintained, is expected to be stronger than that measured in the Fish Pass by a factor of about 1.2. This factor suggests average flow speeds that will be comparable to or exceed 1.5 ft/sec. In summary, because the proposed Packery Channel will be maintained at a depth some two times greater than the actual depth of the Fish Pass, the hydraulic advantage of the deeper Packery Channel compensates for its greater length and frictional loss as compared to the Fish Pass.

The above simple model provides insight into the dynamics of the project channel, but is not adequate for functional design. Therefore, a rigorous two-dimensional model was applied in Report 2 (Brown and Militello 1997) to investigate hydrodynamics (including water surface fluctuations and two horizontal components of the depth-averaged current) associated with re-opening of the channel. Some results of the modeling studies are incorporated below in the inlet stability investigation.

Inlet Stability

In this section we evaluate four methods of estimating the stability of an inlet entrance – investigating whether the inlet will tend to remain open or tend to close.

In most inlet studies, the tidal prism enters as a central parameter governing inlet stability. The tidal prism has been defined slightly differently by various authors, and for this study the tidal prism P is taken to be the average tidal range of a bay times the bay area experiencing that range. Tidal prism is expressed in cubic feet in US customary units. For a one-inlet system, the tidal prism P is a readily calculated quantity that has been correlated with equilibrium cross-sectional areas of tidal inlets with and without jetties and for the different coasts of the United States (Jarrett 1976). The present situation of two inlets, the GIWW, and two water bodies with different depths and tidal ranges makes identification of the tidal prism difficult for representing the gross flow characteristics of the hydrodynamic system.

Tidal Prism Method

With Packery Channel in place, Corpus Christi Bay (including the section of the upper Laguna Madre extending northward from the Kennedy Causeway) would be a two-inlet system, with the much deeper and wider (hydraulically efficient) Corpus Christi Ship Channel "capturing the tidal prism" or flow as compared to the proposed Packery Channel. Applying the same tidal prism to compute a discharge or inlet equilibrium cross-section at both inlets would be double counting. Another complicating factor is the influence of strong northern fronts, which force water to the south; this flow, identified as a major hydrodynamic force for maintaining tidal inlets along the Texas coast (Price 1952, Price and Parker 1979, Suter *et al.* 1982, and others), is not represented in the tidal prism.

In inlet stability studies, the tidal prism is computed for "the bay area experiencing that range (of tide)." At the Packery Channel tide gauge, the range is 0.36 ft, whereas for the tide gauge at the Naval Air Station-Corpus Christi (NAS), the range is 0.56 ft.[13] This difference in range indicates that the terminal basin of the Upper Laguna Madre (Kennedy Causeway to the south margin of Corpus Christi Bay, Figure 47) has a weak hydrodynamic connection to Corpus Christi Bay that diminishes with distance across the shallow lagunal waters. The connection is made weak by the "bulkhead" – a broad, shallow sill stretching across the northeastern end of the Upper Laguna Madre, by various islands, and by the shallower water in the terminal basin of the upper Laguna Madre (2 to 3 ft) as compared to the water in Corpus Christi Bay (10 to 12 ft). Main communication of water between the terminal basin and Corpus Christi Bay is through the GIWW, which has design dimensions of 12-ft depth, 300-ft width at the top, and 125-ft width at the bottom. The GIWW is typically dredged to 14-ft depth.

On the assumption that the bay area for the terminal basin of the Upper Laguna Madre is that which pertains to the proposed Packery Channel entrance (approximately 6.5 sq miles or 1.81×10^8 sq ft), the estimated tidal prism is

$$P = (1.81\times10^8 \text{ sq. ft}) \times (0.36 \text{ ft}) = 6.516\times10^7 \text{ cu ft} \tag{15}$$

This value of a tidal prism is relatively small. According to the study of Jarrett (1976), the equilibrium cross-sectional area of an entrance channel below MSL, A_C, for an inlet with two jetties (as an average for the Atlantic, Gulf, and Pacific coasts) is

$$A_C = 3.76\times10^{-4}\, P^{\,0.86} \tag{16}$$

Application of Eq. (16) with the estimated value of the prism through Eq. (15) gives $A_C = 1.97\times10^3$ sq ft. The proposed Packery Channel, with design entrance channel dimensions of 140-ft bottom width, 11-ft depth (MSL), and 1 on 3 side slopes, has a cross-sectional area of

[13] These numbers may be compared with the tide range at the Bob Hall Pier tide gauge in the Gulf, which is 1.34 ft.

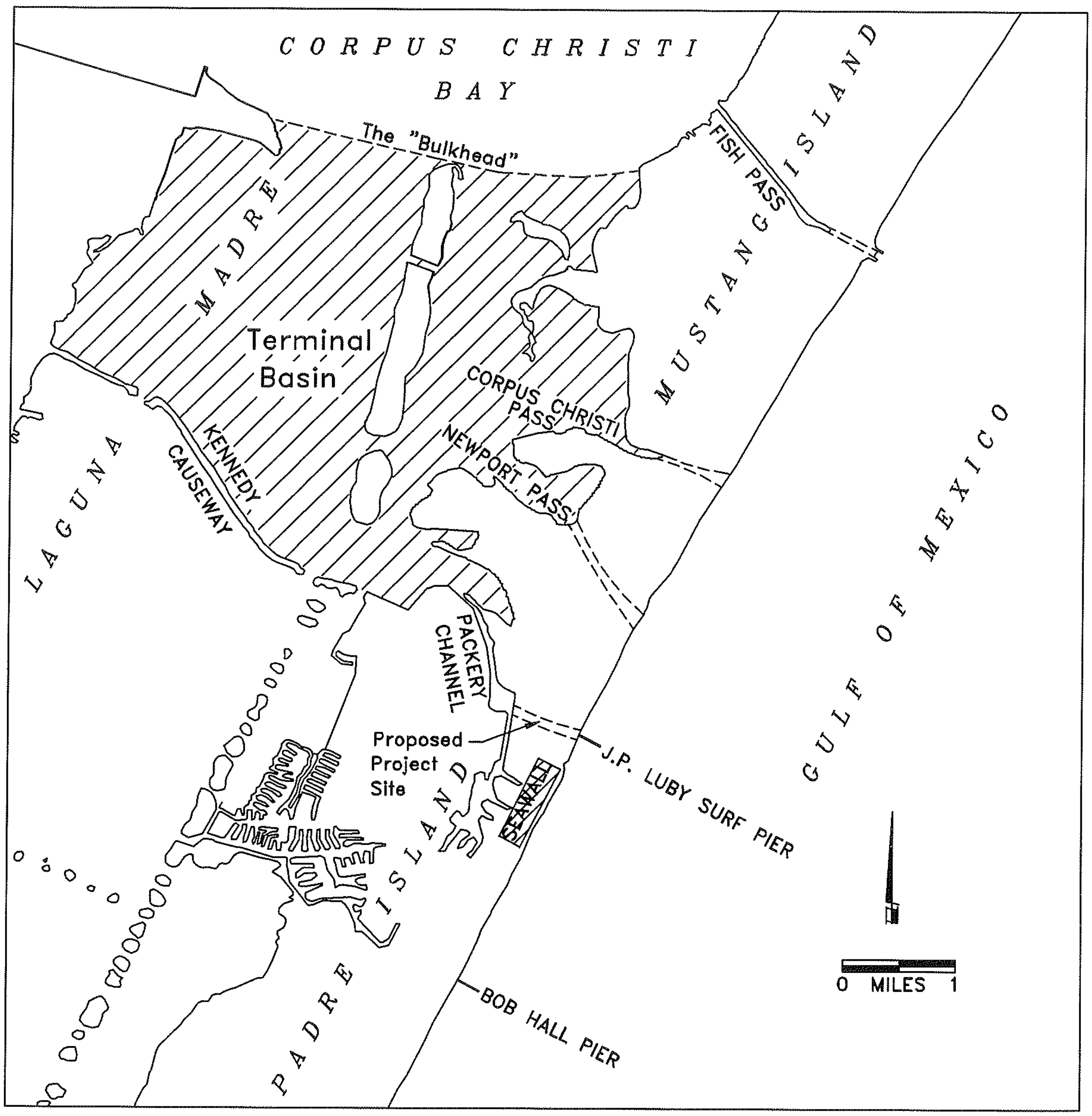

Figure 47. Terminal basin of the Upper Laguna Madre.

1.90×10^3 sq ft. Because the design cross-sectional area approximates the equilibrium area as determined empirically by Jarrett, we expect minimum maintenance requirements for this inlet. In other words, the channel entrance should be self-scouring, at least to the entrance bar that will tend to form. Jarrett (1976) did not have adequate data for inlets with two jetties on the Gulf coast, so the average for the three coasts as given in Eq. (16) was taken as the estimator in the present study.

The result of the above exercise indicates that the channel design is compatible with a simple empirical hydro-morphological relationship, Eq. (16), for preservation of minimum MSL cross-sectional area. The conclusion that the channel will be self-scouring is conservative (erring toward the side of inlet closure) because of the following: (1) The equivalent tidal range in the terminal basin of the Upper Laguna Madre is greater than estimated from the Packery Channel gauge because of the transition to Corpus Christi Bay and its greater tidal range; (2) the stronger flow introduced by the relatively deep GIWW was excluded from the analysis; and, (3) the wind-generated current directed to the south induced by northern fronts was excluded. All of these factors would contribute to greater flow through Packery Channel. On the other hand, the proposed Packery Channel is relatively long and circuitous as compared to most small inlets, with correspondingly greater friction to retard the flow.

Width-to-Depth Ratio Method

Although apparently not much employed in inlet-entrance design, Jarrett (1976) notes that, for a dual-jetty system, the spacing of jetties for promoting stability should be such as to yield a W/R ratio of less than 100, based on his analysis of 108 inlets and 162 associated data points. The interpretation is that deeper and narrower channels are more hydraulically efficient than shallower and broader channels. Taking W/h as an approximation to W/R, we have for the proposed Packery Channel, if maintained, $(W/R)_{PC} \cong 300/11 = 27$, which is considerably less than the cutoff of 100. In contrast, for the Fish Pass, which was not maintained, $(W/R)_{FP} \cong 400/8 = 50$ and $400/2 = 200$, depending on depth selected in the shoaled channel. Table 3 contains width-to-depth ratios for other Texas inlets of interest in this investigation.

Scour Velocity Method

A rule of thumb, as described by Bruun (1990, 1991) and others, states that the maximum flow speed in an inlet entrance should be about 3 ft/sec for promoting if not assuring inlet stability. As described in the previous section and by numerical simulations in Report 2 (Brown and Militello 1997) the depth-averaged flow speed at the entrance will frequently exceed 3 ft/sec. Because the longshore sand transport rate is smaller on the Gulf coast as compared to the Atlantic Ocean and Pacific Ocean coasts, an ebb current frequently reaching 3 ft/sec will tend to maintain the entrance. Riedel and Gourlay (1980) report measurements made at four stable inlets in a wave-sheltered region (region of reduced longshore sand transport) along the coast of Australia. The mean maximum velocity in the inlets was on the order of 1 to 1.5 ft/sec.

Bruun Ratio Method

The preceding did not explicitly include infilling of the channel by wave action and wind. Bruun (1991) discusses inlet stability with focus on a parameter P/M_t introduced by himself and coworkers in the 1960s, where M_t is the total amount of material carried to the entrance in a year. In the present situation, M_t includes longshore transport and wind-blown sand transport. The P/M_t parameter describes a balance between deposition by sediment and the transporting capacity of the inlet, represented by P. According to Bruun (1991), for $P/M_t > 300$, there will be little sediment shoaling or bar seaward of the entrance; for $150 < P/M_t < 300$, there will be limited shoaling and a minor bar; for $100 < P/M_t < 150$ there will be a small entrance bar and only minor navigation problems; for $50 < P/M_t < 100$ there will be a wider and higher bar and increasing navigation problems, and for $P/M_t < 50$, there will be formation of a wide and high bar, and navigation becomes difficult to very difficult.

In the present situation, assuming a probable maximum volume of $M_t = 200{,}000$ cu yd = 5.4×10^6 cu ft, we have $P/M_t = 12$, which indicates that a wide, high bar would tend to form and navigation would be difficult. This analysis indicates that corrective measures should be taken for assuring safe navigation, as discussed in the next section.

The analysis in this section is conservative because the effective tidal prism is expected to be greater than the value used, as described in a previous section, and the gross annual transport rate of 200,000 cu yd is expected to be an overestimate for most years. In addition, the jetties and external spurs will act to prevent the longshore drift from reaching the channel.

Summary of Inlet Stability Estimates

Mustang Island Fish Pass remained open, although becoming very shallow, for about 10 years without dredging maintenance and despite short jetties, a wide entrance, lack of bank protection for several years, and accelerated infiltration by sand blown off mounds of dredged material placed adjacent to the entrance channel. The proposed Packery Channel entrance has a more hydraulically efficient design than the Fish Pass, is located near the original site of the natural southeast corner inlet for Corpus Christi Bay (Price 1952), and is predicted to have flows exceeding those in the Fish Pass (Brown and Militello 1997). Three of the four methods described above for calculating inlet stability, attempting to be on the conservative side (toward channel infilling), indicate that the inlet entrance will tend to be stable.

Therefore, it is concluded that the Packery Channel entrance will not suffer catastrophic instability and will not have a strong tendency for closure. Although the forcing conditions and design parameters place Packery Channel near the dynamic border separating inlet stability and instability, stability can be achieved by reasonable measures to reduce sediment infilling and by maintenance dredging conducted promptly on an as-needed basis, discussed next.

Sedimentation, Inlet Shoals, and Dredging

A channel dredged through a sandy barrier island to connect a bay to the sea (Gulf of Mexico, in this case) will receive sediment (sand, silt, and clay) from several sources and directions. The deposition of sediment is called sediment shoaling or sedimentation. The area of most concern is the channel entrance, where waves are incident and will increase in height and possibly break if the water depth decreases by shoaling. Shoals can also form along the sides of channels, where the transporting capacity of the current is weakest, or in the back bay. Possible sediment sources and directions are listed in Table 6.

Corrective measures for reducing sedimentation in the channel, or decreasing M_t in the context of the previous section, must be taken to keep the inlet open and maintain navigability. Measures discussed in the previous chapter are to: (1) construct spurs to deflect sediment away from the mouth of the channel, and (2) prevent intrusion into the channel by wind-blown sand. Other corrective measures are to: (3) minimize channel bank erosion (by building bulkheads along stretches of the channel with strong currents), (4) collect sediment in deposition basins for

Table 6. Source, direction, and type of material that may be deposited at a channel entrance.

Source	Direction	Material
Longshore transport around ends of jetties	From both lateral sides in and around the surf zone	Sand
Ebb current	From bay	Sand, silt, and clay
Flood current	From Gulf	Sand
Littoral material offshore (from to-and-fro wave motion, coastal and wind-generated currents, etc.)	From offshore and both lateral sides	Sand
Local channel bank slumping and adjustment	From both lateral sides	Sand
Bank adjustment in the channel	From bay (transported by ebb current)	Sand, silt, and clay
Wind-blown sand	From dry beach on both sides of channel	Sand, silt, and clay
Longshore transport through and over the jetties if not impermeable or sufficiently high	From lateral sides in and around the surf zone	Sand

avoidance of bar formation and ease of handling of the deposited material; (5) conduct dredging promptly on an as-need basis; and (6) over-dredge (dredge to greater than design depth).

The potential for sediment to enter the channel is greatest for that brought by the longshore current and by the longshore component of the wind. As discussed in Chapter 2, it is recommended that the jetties extend beyond the seaward-most bar (storm bar) to reduce infiltration into the channel. Spurs will divert the seaward flow of water (rip current) that will run along the jetty away from the channel. The jetties should be sand tightened and sufficiently high to avoid overtopping by water and suspended sediment. Control of sediment movement by spurs and sand-tightened jetties will create deposits that can be hydraulically dredged and bypassed as necessary.

Sand brought by wind will be greatest along the dry, flat beach and on unvegetated dunes, from both the north and the south. Counter measures are discussed in Appendix F and include planting of vegetation, blockage by structures, and high jetties. The wind-blown sand deposits must also be managed and enter in the bypassing and sand redistribution plan.

To maintain a channel depth of at least 10 ft MLLW (11 ft MSL), it is recommended that the entrance area of the channel, where sediment will be deposited, be over-dredged by 2 ft or more to provide a deposition basin or collection area. Because mobilization constitutes a substantial portion of dredging costs, over-dredging and advance dredging will reduce dredging frequency and cost. Because ocean-going dredging equipment may not be able to reach the relatively shallow design depth of 11 ft MLLW, over-dredging may give substantial cost savings.

The inner basin (see Figure 2) should also be over-dredged to form a deposition basin for sediments brought from the Gulf and from the channel leading from the SH361 bridge. The greater water volume in the basin decreases the current speed, as demonstrated in the hydrodynamic calculations described in Report 2 (Brown and Militello 1997). The inner basin can serve as a deposition basin for convenient dredging in an environment protected from waves, reducing down time and size of equipment needed, and, therefore, the cost of dredging. The Fish Pass did not benefit from the presence of an inner deposition basin.

In conclusion, dredging requirements and bypassing schedule will depend mainly on the functioning of the spurs and deposition basin and on mitigation measures for wind-blown sand, as well as on the wave climate for any given year. Longshore sand transport can bring sand to the location of a channel in both a gradual fashion and as the sudden arrival of a large slug of sand, as from the break up of a shoal or bar that might form near the spurs and move under high-energy waves and current of a storm. Under the assumption that most of the wave-and wind-transported sand will probably be blocked by the jetties or otherwise mitigated and periodically bypassed as necessary, the mechanically bypassed volume is estimated as 50,000 ± 40,000 cu yd as an annual average corresponding to the average net transport. This estimate is in accord with the average magnitude and standard deviation of the net longshore sand transport rate calculated through shoreline change modeling.

With the majority of potentially infiltrating sand blocked or collected in the inner deposition basin, dredging requirements for the mouth of and along the entrance channel will be relatively minor. An upper limit for the total average annual volume of material that may be expected to enter the entrance channel by naturally bypassing the jetties from the north and south, by slumping of the channel walls, by to-and-fro transport offshore, and by scour of the channel itself, is estimated to be a similar amount, on the order of 50,000 ± 10,000 cu yd. This estimate is based on a percentage of the average magnitude and standard deviation of the gross longshore sand transport rate calculated through shoreline change modeling. The sand dredged from the entrance channel is expected to be of beach quality and should be placed on the downdrift beach together with the mechanically bypassed sand.

The required volumes of sand to be mechanically bypassed and dredged from the entrance channel total 100,000 ± 50,000 cu yd/yr, a volume that is about 60 percent that dredged at Port Mansfield on average over a 30-year period. The predicted upper limit of total maintenance-dredging volume of 150,000 cu yd/yr is realistic, considering that the proposed Packery Channel is located near to a null point in longshore transport (which reduces mechanical bypassing requirements) and because appropriate measures will be taken to reduce natural sand bypassing and infiltration by wind-blown sand. Provision should be made in planning for emergency dredging after storms.

Impacts to the Corpus Christi Ship Channel

An obvious implication of the introduction of another permanent tidal inlet to Corpus Christi Bay is the diversion of some of the flow from the Corpus Christi Ship Channel and Aransas Pass to the new inlet. According to Turner, Collie & Braden, Inc. (1967a), this potential implication at the Fish Pass resulted in early concern by the USACE, which "informally" indicated that the permit for the proposed Pass would not be granted if the Pass was of such magnitude to impact flows at Aransas Pass. The Fish Pass, therefore, was designed to accommodate only small-boat navigation. Behrens *et al.* (1977) suggested that "a basic conflict in design rationale (therefore) existed," because the Pass could not be designed to both enhance flushing of the Bay and to prevent tidal discharge from impacting the stability of Aransas Pass. However, Turner, Collie & Braden, Inc. (1967a) recognized that opening the Pass alone could not eliminate the problem of high bay-water salinity, but believed that the Pass would be a "pivotal element in any <u>system</u> of improvements designed to cope with...hypersalinity." The details of this system are not given.

The proposed Packery Channel should not significantly impact the flows through Aransas Pass because the channel is approximately equal in cross-sectional area to the original Fish Pass. The proposed channel cross-sectional area is only 7% of that at Aransas Pass[14] and, as discussed earlier in this chapter, probably will not serve the same tidal basin as Aransas Pass. Note also that maintenance of Aransas Pass resulted in the closure of the historic Packery Channel in the 1930s, which indicates that Aransas Pass is now the dominant pass for the bay system of Corpus Christi Bay and the Upper Laguna Madre. Because the cross-sectional area of Aransas Pass is 10- to 15-times greater than would be at Packery Channel, the relative discharge of water would be only a few percent.

Wave-Current Interaction

Waves arriving at an inlet entrance will meet an ebb current (opposing current), a flood current (following current), or no current depending on the stage of the tide and strength and direction of the wind. The depth-averaged tidal and wind-generated current in the channel entrance was computed in Report 2 (Brown and Militello 1997). At the channel entrance, to a good approximation this current will appear as a uniform flow to the incident surface waves and will be treated as such in this section. A current, in particular an opposing current, will increase the height and decrease the length of the incident waves at the channel so as to steepen the wave. Wave steepening increases as the speed of the opposing current increases. This section makes an estimate of wave changes within the Packery Channel entrance channel to assess the consequences for navigation safety.

Wave steepness is defined as the ratio of the wave height H to wavelength L at a given location. This ratio H/L can be thought of as a rise over a run in distance, similar to a slope. Wave steepness can vary widely; for example, a value $H/L = 0.001$ corresponds to a very gentle or flat wave, called "swell," whereas a value of 0.05 corresponds to a steep storm wave, wind chop, or "sea." The Gulf of Mexico has steeper waves than the Atlantic or Pacific Oceans because the period T of the waves, and, therefore, the wavelength, is short compared to waves in larger water bodies.

Waves will break if their steepness exceeds a certain value that can be estimated with the Miche formula (Lé Mehauté 1976),

$$H_{\max} = 0.14\, L \tanh\left(\frac{2\pi h}{L}\right) \tag{17}$$

[14] The dimensions of the Aransas Pass Entrance Channel were provided through personal communication with Paul D. Carangelo, The Port of Corpus Christi Authority, September, 1996. These dimensions are as follows: bottom width of 500 ft, a maintained depth of 47 ft mean tide level (includes 2 ft of over dredging), and 1-on-2 side slopes.

where H_{max} is the maximum wave height allowable for a wave of length L in water of depth h. In deep water, $\tanh(2\pi h/L) = 1$, so that the maximum possible wave steepness is $H_{max}/L = 0.14$. In very shallow water, the Miche formula gives $H_{max} = 0.88\ h$, with the 0.88 factor close to the value of 0.78 typically specified for depth-limited breaking waves. In Eq. (17), the quantity L is modified if the wave is moving on a current.

The calculation of wave height and wavelength in the presence of a current in water of arbitrary depth is an advanced topic, and rigorous treatment proceeds from the conservation of a quantity called "wave action" (see Jonsson 1990). As an example of the change in wave height and length in the presence of an opposing current, it is assumed that the depth-averaged current V in the channel is uniform, the depth h in the channel is constant, and shallow-water linear-wave theory is applicable. Then the following formulas are applicable for estimating the height H_E and the length L_E of a wave in the current at the channel entrance, where H and L are the corresponding values just offshore of the entrance, beyond the influence of the current:

$$H_E = \frac{1}{1 - \dfrac{V}{\sqrt{gh}}} H \tag{18}$$

and

$$L_E = (\sqrt{gh} - V)T \tag{19}$$

In writing these formulas, an opposing or ebb current was assumed. If a following current were to be considered, the minus sign in each equation should be changed to a plus sign. For an opposing current, the wave height increases (Eq. (18)) and the wavelength decreases (Eq. (19)), thereby increasing the wave steepness. The period T is that associated with the waves beyond the influence of the current. In the above $(gh)^{1/2}$ is the speed or celerity of the wave, where g is the acceleration due to gravity (32.2 ft/sec^2).

In the following, we assume a depth $h = 10$ ft, wave height of 2 ft, and a wave period $T = 5$ sec just beyond the influence of the channel current. Such a wave condition corresponds to a typical sea that boats would be expected to encounter routinely. In the absence of a current ($V = 0$), the wavelength $L = T\sqrt{(gh)} = 90$ ft, approximately, for this example. Therefore, for $H = 2$ ft, the steepness is $H/L = 2/90 = 0.022$ for this typical wave on the Texas coast. In the presence of an opposing current of 3 ft/sec, by Eqs. (18) and (19) the wave height and wavelength become $H_E = 2.4$ ft and $L_E = 75$ ft, respectively, giving a steepness $H_E/L_E = 0.032$. This example is only illustrative to show the physical processes, because the conditions stated only approximate waves in shallow water.

The wave height, wavelength, and steepness for an ebb current of 1 ft/sec and 3 ft/sec at the inlet entrance, found in Report 2 to be expected typical weaker and stronger flows, respectively, were calculated as listed in Table 7. In developing the values in Table 7, a rigorous solution of the wave-action equation for arbitrary depth was employed, and not the shallow-water approximations given above. The typical ebb current of 1 ft/sec increases the wave height by 9 %, decreases the wavelength by about 7 %, and increases the wave steepness by 19 % with respect to the no-current or base condition. The stronger ebb current increases the wave height by 37 %, decreases the wavelength by 23 %, and increases the wave steepness by 81 % (almost doubles the steepness) with respect to the base condition. An increase in the steepness of high waves approaching a strong ebb current can pose a hazard to navigation. Wave steepening is commonly observed at all tidal inlet entrances and can be minimized by maintaining the navigation channel to design depth.

Table 7. Wavelength and steepness for waves of three heights and period of 5 sec (h = 10 ft).

No Current (base condition)			V = 1 ft/sec			V = 3 ft/sec		
H, ft	L, ft	H/L	H_E, ft	L_E, ft	H_E/L_E	H_E, ft	L_E, ft	H_E/L_E
1	82.4	0.012	1.09	76.4	0.014	1.37	63.5	0.022
2	82.4	0.024	2.18	76.4	0.029	2.74	63.5	0.043
3	82.4	0.036	3.27	76.4	0.043	4.11	63.5	0.065

If the channel is allowed to shoal substantially, the current will become stronger because of the constricted cross-section at the entrance, and at the same time the wave speed $(gh)^{1/2}$ will become smaller because h, the depth, is less. Therefore, H_E will increase and L_E will decrease substantially, producing waves of greater steepness that might pose a threat to boating. If the wave height at the entrance approaches the value 0.78 h (depth-limited breaking) or if the ebb current velocity approaches the speed of the wave (current-limited breaking or "wave blocking"), then the waves will break on the shoal and definitely pose a danger to boating. These considerations apply to all inlets and entrances that experience tidal currents and waves.

The Miche equation (Eq. (17)) gives maximum possible wave heights of 7.41, 7.23, and 6.73 ft for the base condition, 1- , and 3-ft/sec ebb currents, respectively. The maximum wave height decreases as the speed of the opposing current increases; in other words, waves will break more readily on an opposing current. These maximum wave heights are considerably greater than the corresponding heights given in Table 7. However, if the incident wave height of a 5-sec wave is about 6 ft in deep water, then this wave would be expected to break in the channel in the presence of a stronger ebb current.

For completeness, we consider a 3-ft wave with 5-sec period approaching directly to the proposed Packery Channel entrance with a 1-ft/sec flood current. This wave will have its height reduced to 2.78 ft, its wavelength increased to 88.2 ft, and its steepness decreased to 0.032 as compared to the corresponding base condition. The flood current reduces the wave height and wave steepness, favoring navigation.

5. Summary and Conclusions

The proposed Packery Channel would reopen the historic inlet to the Gulf for Corpus Christi Bay and would be well situated for stability – in a region of low net longshore sand transport and in the southeast corner of the bay. The relatively small net longshore transport of sand would need to be maintained through mechanical sand bypassing to minimize impacts of the channel entrance on the downdrift shoreline. The Mustang Island Fish Pass, an artificial inlet dredged between the Gulf and Corpus Christi Bay that was created in 1972 just 5 miles north of the proposed channel, remained open for about 10 years without maintenance dredging despite having short, ineffectual jetties. Analysis of historic dredging records at Mansfield Pass, an artificial inlet located 70 miles to the south and which is dredged deeper and is less hydraulically efficient than the proposed Packery Channel, provides an estimate of expected annual dredging maintenance volume for Packery Channel of 175,000 ± 50,000 cu yd. A more realistic estimate of required annual dredged volume at Packery Channel is 100,000 ± 50,000 cu yd based on wave hindcast and shoreline change modeling, the smaller design depth of 11 ft as compared to 16 ft or 26 ft at Mansfield Pass, anticipated effectiveness of the proposed spur jetties at Packery Channel, and maintenance of effective jetty length through beach-management practice. Sand dredged to cut the channel will be available to nourish adjacent beaches and, therefore, temporarily reduce threats associated with localized erosion.

The proposed Packery Channel also holds the advantage over the Fish Pass and Mansfield Pass of having an inner basin to serve as a deposition basin for trapping sediments to reduce sedimentation at the entrance mouth and at the bay side of the channel. The deposition basin can be conveniently dredged. Outside spurs proposed for the jetties would reduce the rate of infiltration of sand to the entrance and tend to stabilize the position of the shoreline near the jetties. Infiltration of wind-blown sand to the channel, identified as being a significant factor for closure of the Fish Pass, could be controlled at the proposed Packery Channel, further reducing sedimentation in the channel. The combined tidal and wind-generated current in the channel entrance was found to have moderate strength and not pose a hazard to boating if the channel is maintained to design depth. Packery Channel will not have an adverse impact on the Corpus Christi Ship Channel because the proposed channel will be significantly smaller and will not serve the same tidal basin. All of the forgoing can be viewed as favorable aspects of the proposed Packery Channel.

There are negative aspects or certain long-term costs of the Packery Channel project as well. The channel will have to be maintained at an average rate of approximately 100,000 ± 50,000 cu yd. If construction follows the proposed functional design, and, in particular, if the entrance is over-dredged to a depth of 13 to 14 ft MSL instead of the design depth of 11 ft MSL, dredging is not expected to be necessary for the first few years after project completion. During this time, the shoreline will come to equilibrium with the jetties. Thereafter, dredging will be required at intervals of 1 to 3 years, depending on weather patterns (waves and wind), equilibration of the channel, and deposition pattern of the sediment. The required maintenance dredging operations include the mechanical bypassing of sand past the jetties, which will minimize project impacts on downdrift beach erosion. Although the recommended project design and beach management probably would have a minimal contribution to beach erosion, the project would be located on a coast that is eroding – primarily by inland transport of sand by wind and by storm washover, with a component of apparent erosion caused by relative sea-level rise. On a regional scale, the recession of the shoreline will continue because the sediment supply is limited, and some sand may move from the beach to form an ebb-tidal shoal.

The following is a summary of the recommended basic functional design and accompanying sand-management practices developed in this study. The recommendations were arrived at based on review of the literature of the physical processes at the site, field data collection, numerical modeling of shoreline change and inlet hydrodynamics, comparison with neighboring inlets, and records of the behavior of inlets around the coasts of the United States.

1. The jetties should be impermeable and extend at least 1,400 ft from the position of the MSL shoreline. A 12-deg orientation north from shore-normal will provide sheltering from southeast waves that are common in summer. Outside spurs near the ends of the jetties will reduce transport of sediment into the channel by redirecting the material away from the entrance. The spurs will also reduce possible recession of the shoreline adjacent to the jetties.

2. The design channel depth is 11 ft MSL (10 ft MLLW), with a 140-ft bottom width, 1-on-3 channel side slopes, and 300 ft between jetties.

3. The deposition basin east of the SH361 bridge and the inlet entrance should be over-dredged to reduce long-term maintenance cost by reduction of dredging frequency. Over-dredging at the Gulf entrance to 13-ft MSL is desirable and expected because of draft requirements of ocean-going dredges. Over-dredging will promote navigation safety by decreasing the speed of the ebb current and its steepening of waves, and by reducing potential for depth-limited wave breaking.

4. Minimization of wind-blown sand intrusion into the channel is essential and is a cost-effective means of reducing maintenance dredging volume.

5. Maintenance dredging will be an integral part of the long-term cost of Packery Channel. The estimated average annual dredging volume is 100,000 ± 50,000 cu yd. The sand dredged

from the entrance channel and from the deposition basin should be placed on the downdrift beach.

6. The ebb current in the inlet entrance will steepen waves, as is the case at all tidal inlets. For waves of 5-sec period, breaking in the channel for stronger ebb current is expected only for incident waves with heights greater than about 6 ft. Because these breaking waves will prohibit safe navigation, small-craft advisory warnings are expected to discourage boating during this sea state.

7. The adjacent beaches should be monitored through periodic beach profile surveys and aerial photography to document the impact of the jetties and to compensate through mechanical sand bypassing. Mechanical bypassing probably will not be necessary during occasional short-term (1-to-2 year) net transport reversals, depending on wave climate.

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Appendix A : Packery Channel Permit

This appendix contains the amended Permit 18344 as submitted by Fish Trackers Inc./Reopen Packery Channel Association to the USACE, Galveston District.

DEPARTMENT OF THE ARMY
GALVESTON DISTRICT, CORPS OF ENGINEERS
P.O. BOX 1229
GALVESTON, TEXAS 77553-1229

REPLY TO ATTENTION OF

SEP 29 1994

Special Actions Section

SUBJECT: Permit No. 18344(01); Modification

Fish Trackers Inc./Reopen Packery
Channel Association
P.O. Box 4746
Corpus Christi, Texas 78469

Gentlemen:

Your request to amend Permit 18344 to reopen and maintain Packery Channel is approved. The amendment consists of extending the time to complete the work to December 31, 1997. The permit has also been amended to delete all commercial development on the north side of the channel. The 200,000 cubic yards of dredge material which was proposed as fill for the commercial areas will now be placed on the beach for nourishment purposes. The project is located in Packery Channel and the Gulf of Mexico, between Padre Island and Mustang Island, Nueces County, Texas.

The original permit dated, June 14, 1988, provided authorization to hydraulically dredge and bulkhead a 300- by 4500-foot channel, to place dredged material on the south side of the channel and on the beach in front of the Padre Island seawall, to construct jetties and a sand transfer system, to relocate an existing pier, and to construct additional amenities including navigational aids, roadways, parking areas, and walkways.

The enclosed plans in seven sheets, dated September 1994, will now supersede the original permit plans in eight sheets, dated September 1991. All conditions of the original permit remain in full force and effect. In addition, the following special conditions are added to the permit:

g. Any material changes in or a failure to implement the FWS Compliance Agreement dated August 27, 1994 will be grounds for modifying, suspending, or revoking this authorization.

h. That the work will be performed in compliance with the enclosed water quality certification from the Texas Natural Resource Conservation Commission.

FOR THE DISTRICT ENGINEER:

MARCOS DE LA ROSA
Chief, Regulatory Branch

Copies Furnished:

Commander, Eighth Coast Guard District, New Orleans, LA

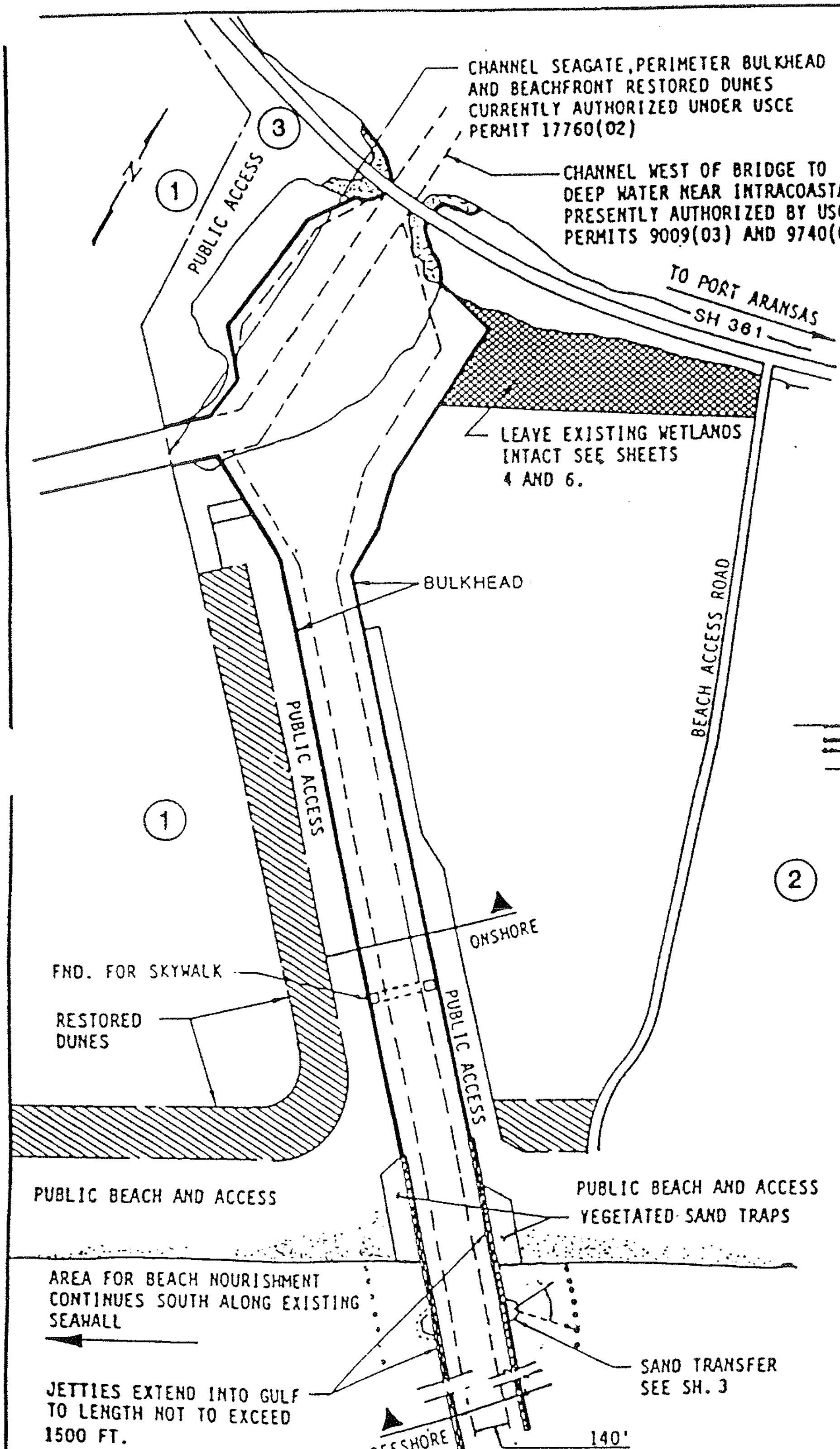
NOAA/NOS, Silver Springs, MD

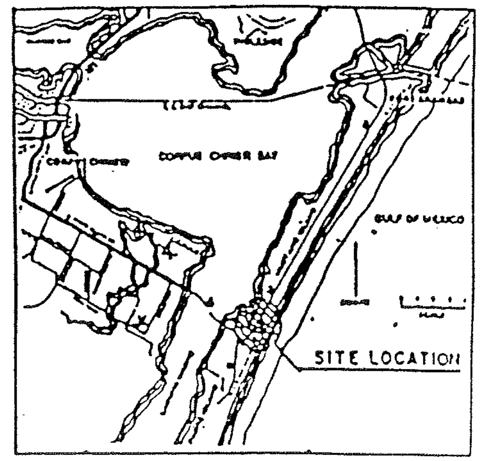
Texas General Land Office, Austin, TX

Texas General Land Office, Aransas Pass, TX

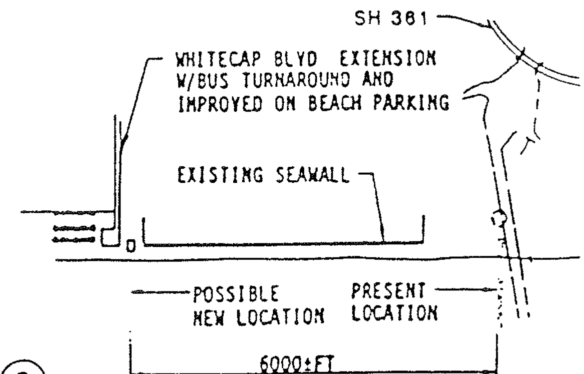
Shiner, Moseley and Associates, Inc., Corpus Christi, TX

Corpus Christi Area Office, Corpus Christi, TX





LOCATION MAP



RELOCATION OF SURFING PIER BEING DISPLACED BY CHANNEL REOPENING

SEE SH 7 FOR DETAILS

SPECIAL PERMIT CONDITIONS

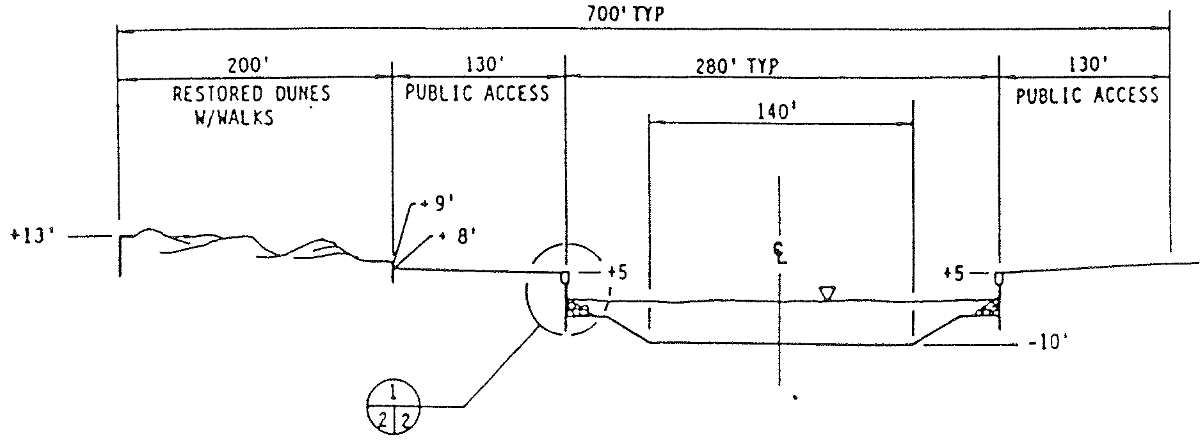
1. Permittee will provide slope protection along the embankments of the State Highway 361 bridge over Packery Channel in accordance with a plan to be approved by the Texas Department of Transportation.
2. Permittee agrees to provide a small opening through each jetty in the nearshore zone to allow for the passage of marine/estuarine species, with such opening being located and designed in a way to be compatible with the sand transfer system and beach maintenance program.

GRAPHIC SCALE IN FEET

400 0 400 800

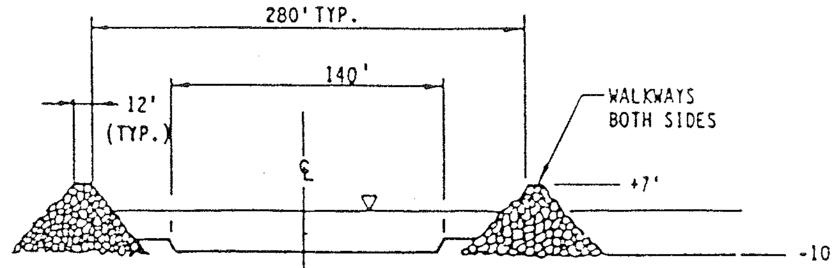
SHINER, MOSELEY AND ASSOCIATES
ENGINEERS + CONSULTANTS

PURPOSE: REOPEN PACKERY CHANNEL TO PROVIDE SMALL BOAT ACCESS TO GULF, CONSTRUCT A SMALL BOAT BASIN, TO INCREASE WATER EXCHANGE WITH LAGUNA MADRE THEREBY IMPROVING AQUATIC HABITAT, RELOCATE SURFING PIER DATUM: MEAN SEA LEVEL ADJACENT PROPERTY OWNERS: 1 LAKE PADRE LTD. 2. TIPTON GROUP 3. STATE OF TEXAS.	PROPOSED: DREDGE CHANNEL, CONSTRUCT BULKHEADS AND JETTIES, INSTALL SAND BYPASS SYSTEM, AND PILINGS COUNTY: NUECES STATE: TEXAS APPLICANT: FISH TRACKERS INC. / REOPEN PACKERY CHANNEL ASSOC. SHEET 1 OF 7 DATE: 9/91 REV. 9/94



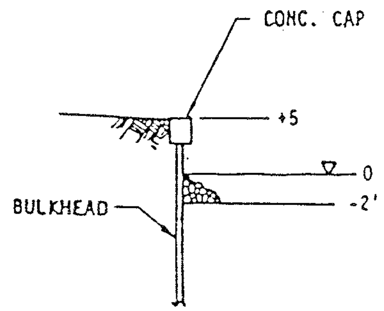
1. ARMOUR PLATING IN THE FORM OF MATS OR RIPRAP MAY BE PLACED AT TOE OF BULKHEAD FOR SCOUR PROTECTION
2. TIMBER MOORINGS MAY BE CONSTRUCTED AT SELECT LOCATIONS ALONG BULKHEAD OUTSIDE OF CHANNEL; TOTAL LENGTH NOT TO EXCEED 20% OF BULKHEAD DISTANCE

DETAILS OF TYPICAL ONSHORE SECTION

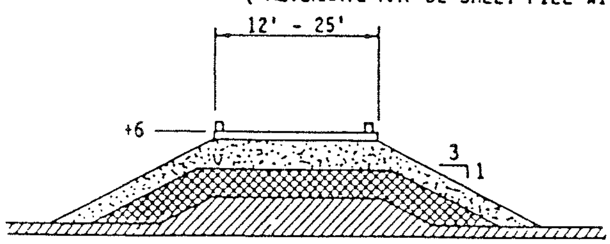


DETAILS OF TYPICAL OFFSHORE SECTION

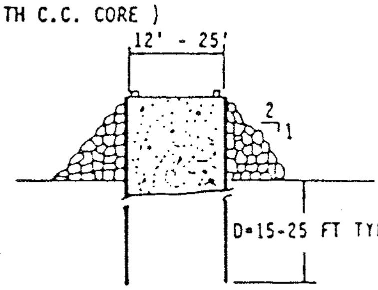
(ALTERNATE MAY BE SHEET PILE WITH C.C. CORE)

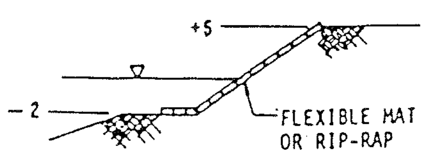


1/2 DETAIL



RUBBLE MOUND





COVER STONE: 4-10 TON GRANITE OR LIMESTONE

MIDDLE LAYER: 0.5-2 TON LIMESTONE

MAT/CORE: 0.5"-200# LIMESTONE

SHEET PILE DESIGN

STEEL SHEET PILE
CONCRETE CORE
LIMESTONE RIPRAP FOR SCOUR PROTECTION

ALTERNATE SHORELINE PROTECTION FOR NORTH SIDE OF CHANNEL

TYPICAL JETTY DETAILS

JETTY CONSTRUCTION MAY BE EITHER RUBBLE - MOUND DESIGN, OR SHEET/PILE, COFFERDAM DESIGN, OR SOME COMBINATION OF THE TWO DESIGNS

SHINER, MOSELEY AND ASSOCIATES
ENGINEERS + CONSULTANTS

PURPOSE: REOPEN PACKERY CHANNEL TO PROVIDE SMALL BOAT ACCESS TO GULF, CONSTRUCT A SMALL BOAT BASIN, TO INCREASE WATER EXCHANGE WITH LAGUNA MADRE THEREBY IMPROVING AQUATIC HABITAT, RELOCATE SURFING PIER

DATUM: MEAN SEA LEVEL

ADJACENT PROPERTY OWNERS: SEE SHEET 1

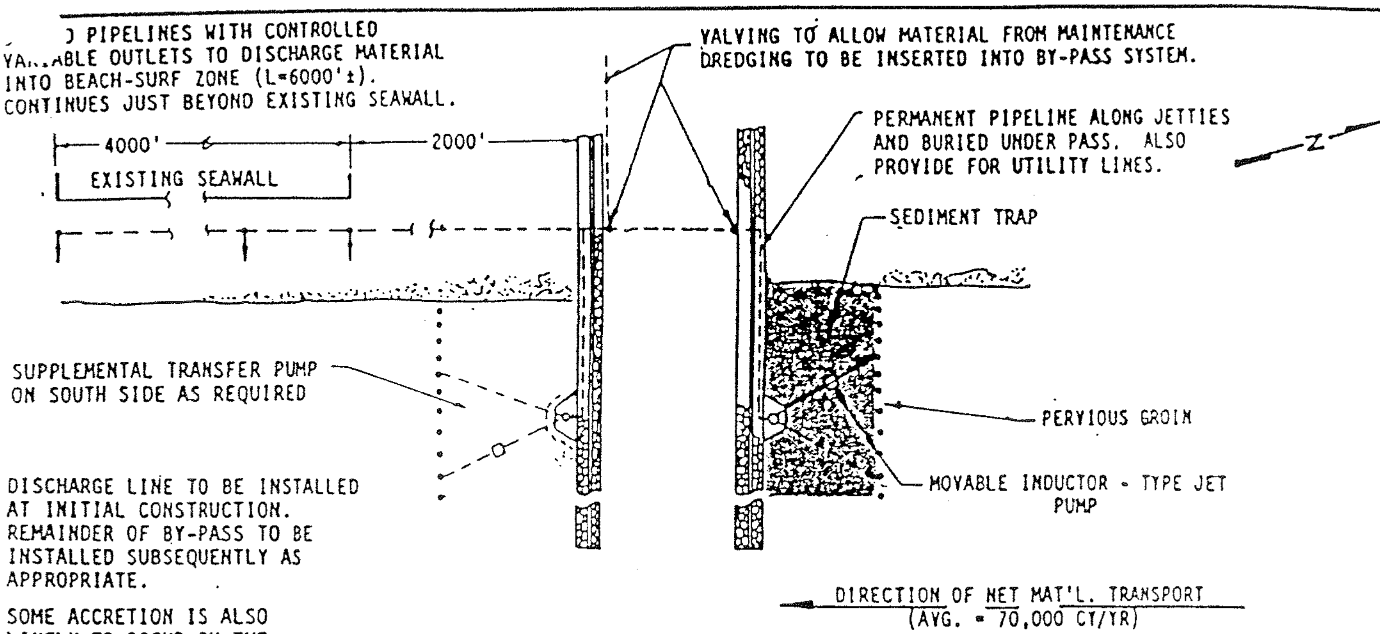
PROPOSED: DREDGE CHANNEL, CONSTRUCT BULKHEADS AND JETTIES, INSTALL SAND BYPASS SYSTEM, AND PILINGS

COUNTY: NUECES STATE: TEXAS

APPLICANT: FISH TRACKERS INC. / REOPEN PACKERY CHANNEL ASSOC.

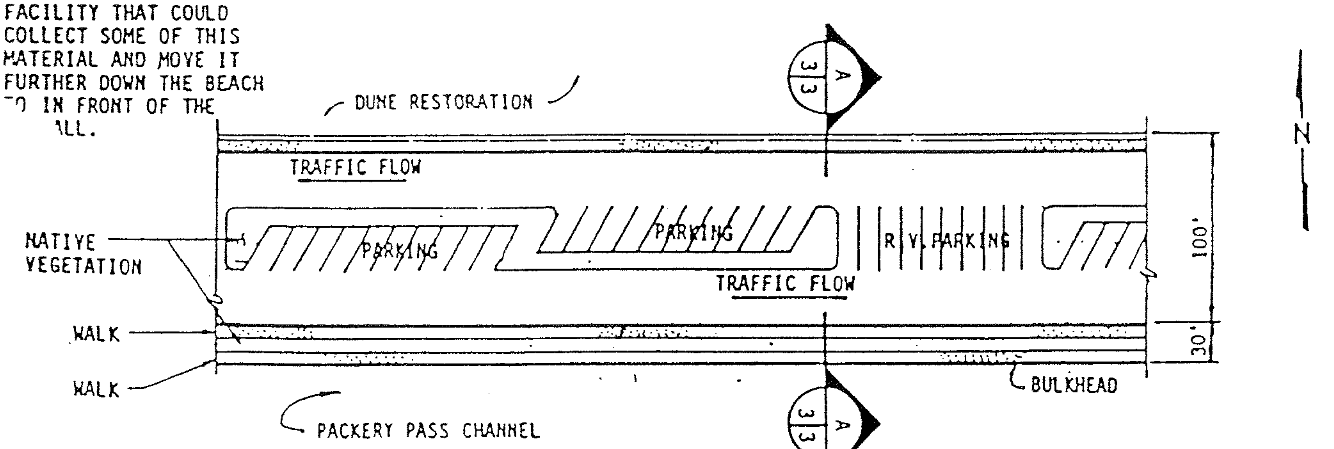
SHEET 2 OF 7

DATE: 9/91
REV. 9/94



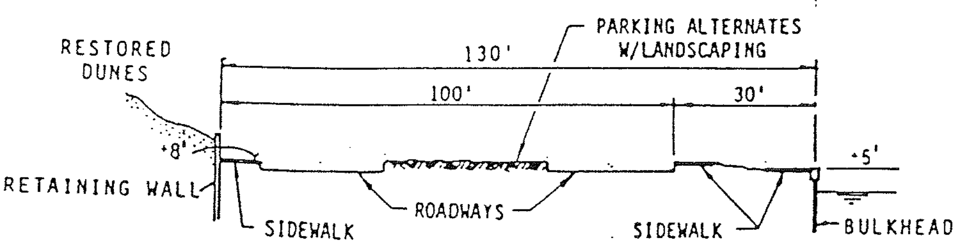
SOME ACCRETION IS ALSO LIKELY TO OCCUR ON THE DOWNDRIFT SIDE. THUS IT IS DESIRABLE TO ALLOW FOR A SAND TRANSFER FACILITY THAT COULD COLLECT SOME OF THIS MATERIAL AND MOVE IT FURTHER DOWN THE BEACH TO IN FRONT OF THE ...ALL.

SAND TRANSFER SYSTEM



TYPICAL PARKING PLAN FOR
PUBLIC ACCESS ALONG CHANNEL

NOTE: AVG. TOTAL SPACE REQD. FOR PARKING AS SHOWN, INCLUDING GRASS =12'; FOR APPROX. 3850' LF AS SHOWN ON SHEET 1, THIS WOULD PROVIDE 320 PARKING SPACES.



SECTION A-A

DETAILS OF STREET AND WALKS MAY VARY

SHINER, MOSELEY AND ASSOCIATES
ENGINEERS + CONSULTANTS

PURPOSE: REOPEN PACKERY CHANNEL TO PROVIDE SMALL BOAT ACCESS TO GULF, CONSTRUCT A SMALL BOAT BASIN, TO INCREASE WATER EXCHANGE WITH LAGUNA MADRE THEREBY IMPROVING AQUATIC HABITAT, RELOCATE SURFING PIER

DATUM: MEAN SEA LEVEL

ADJACENT PROPERTY OWNERS: SEE SHEET 1

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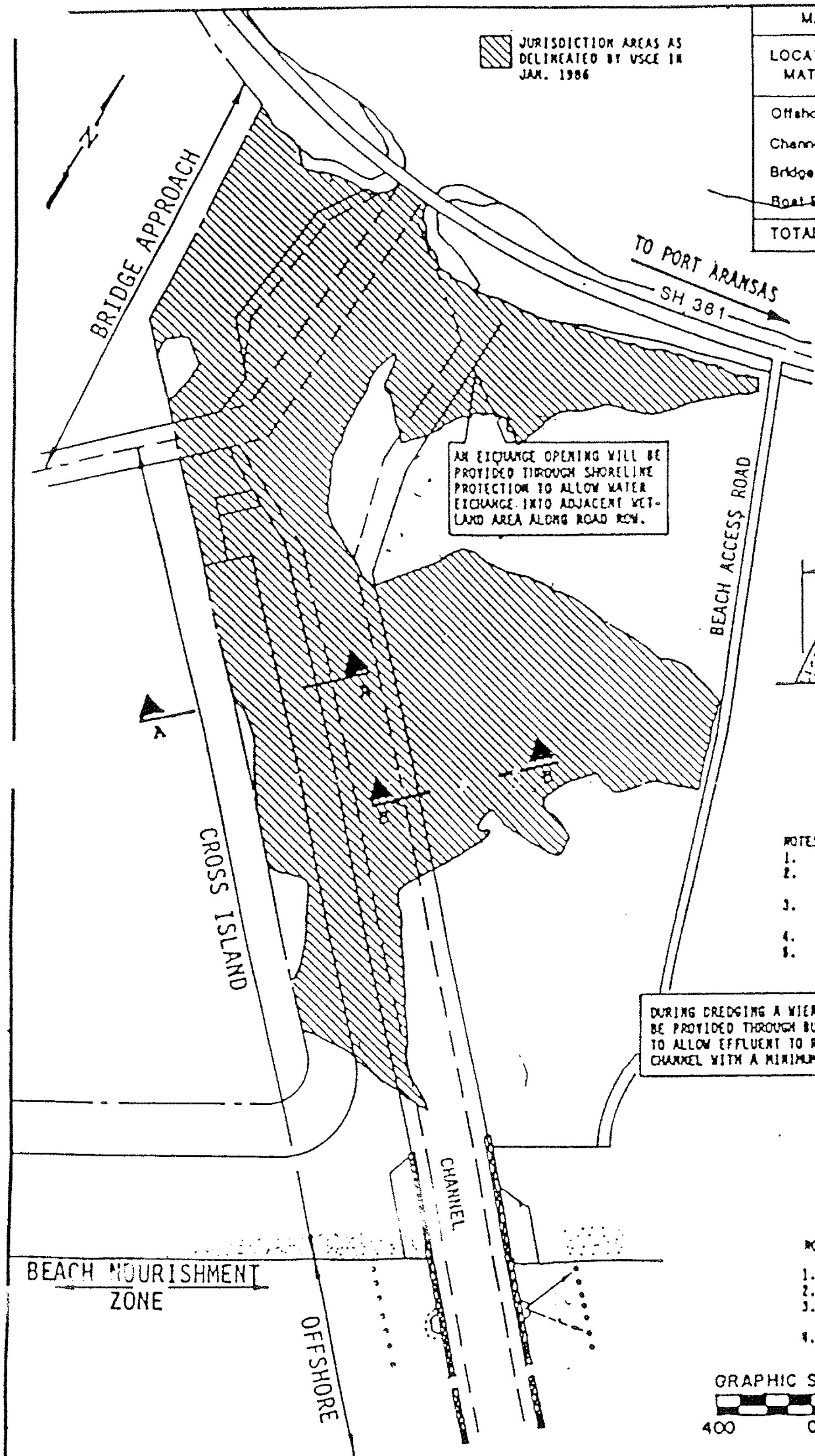
COUNTY: NUECES STATE: TEXAS

APPLICANT: FISH TRACKERS INC./ REOPEN PACKERY CHANNEL ASSOC.

SHEET 3 OF 7

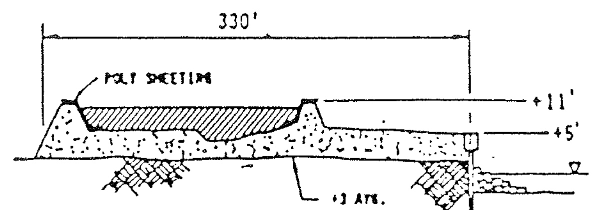
DATE: 9/91
REV. 9/94



MATERIAL SOURCE		DISPOSITION OF MATERIAL		
LOCATION OF MATERIAL	VOLUME TO BE REMOVED (CY)	PUBLIC ACCESS DUNE AREAS SO. SIDE	NO. SIDE	BEACH
Offshore	20,000	–	–	20,000
Channel	260,000	30,000	45,000	185,000
Bridge Approach	280,000	230,000	–	50,000
~~Boat Basin~~	~~195,000~~	–	–	195,000
TOTALS	755,000	260,000	45,000	450,000

MATERIAL BALANCE FOR DREDGING

Total quantities to remain constant; some variation may occur between specific sites to allow for most efficient construction sequence.

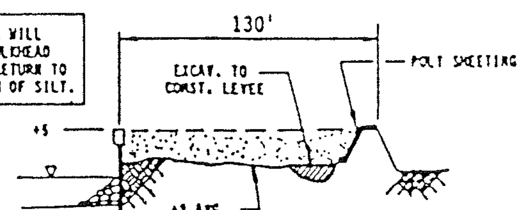


SECTION A-A

TYPICAL SOUTH SIDE OF CHANNEL
AS SEEN DURING CONSTRUCTION

SEE SH 2 FOR FINAL CONFIGURATION

NOTES
1. BULKHEAD ALONG CHANNEL WILL BE CONSTRUCTED FIRST.
2. INITIAL DISPOSAL WILL BE PLACED BEHIND BULKHEAD TO BRING ELEV. UP TO +5'.
3. CONTAINMENT LEVEES TO BE BUILT FROM THIS MATERIAL (COARSE SAND, WORK WITHIN 7 DAYS).
4. ADDITIONAL MATERIAL TO BE PLACED INSIDE LEVEES.
5. FINAL SURFACE TO BE REWORKED INTO DUNES AND PUBLIC ACCESS AS SHOWN ON SH. 2.



SECTION B-B

TYPICAL NORTH SIDE OF CHANNEL
AS SEEN DURING CONSTRUCTION

NOTES
1. BULKHEAD WILL BE BUILT FIRST.
2. LEVEE TO BE CONSTRUCTED FROM ADJACENT MATERIAL AS SHOWN.
3. DREDGE MATERIAL FROM CHANNEL TO BE PLACED BETWEEN LEVEE & BULKHEAD AS SHOWN.
4. SURFACE TO BE FINISHED TO PROVIDE PUBLIC ACCESS AS SHOWN ON SH. 2.

GRAPHIC SCALE

400 0 400

SHINER, MOSELEY AND ASSOCIATES
ENGINEERS + CONSULTANTS

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DATUM: MEAN SEA LEVEL

ADJACENT PROPERTY OWNERS: SEE SHEET 1

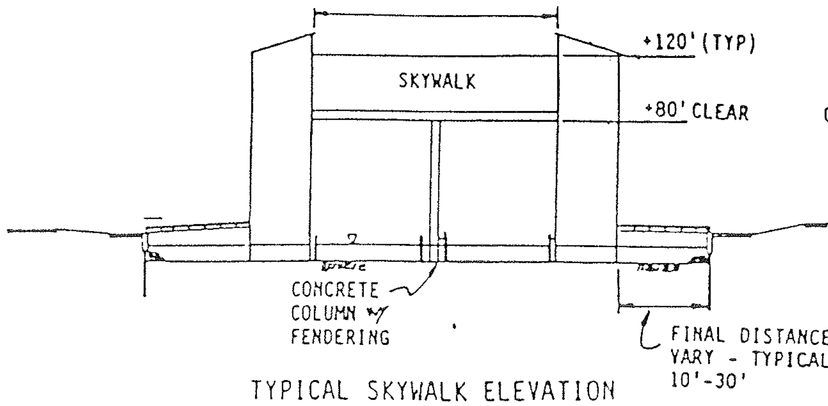
PROPOSED: DREDGE CHANNEL, CONSTRUCT BULKHEADS AND JETTIES, INSTALL SAND BYPASS SYSTEM, AND PILINGS

COUNTY: NUECES STATE: TEXAS

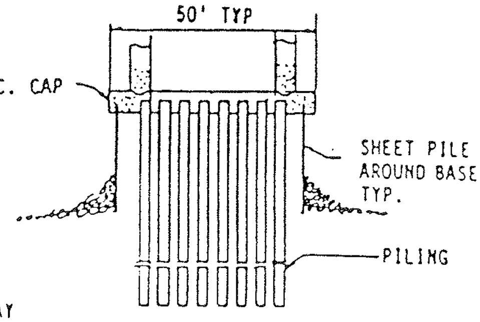
APPLICANT: FISH TRACKERS INC. / REOPEN PACKERY CHANNEL ASSOC.

SHEET 4 OF 7

DATE: 9/91
REV. 9/94



TYPICAL SKYWALK ELEVATION



TYPICAL SKYWALK FOUNDATION

NOTE: ALL CONSTRUCTION IN THIS AREA DELETED FROM ORIGINAL PERMIT.

SHINER, MOSELEY AND ASSOCIATES
ENGINEERS + CONSULTANTS

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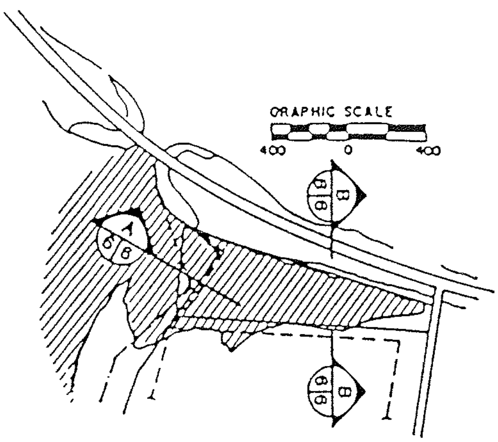
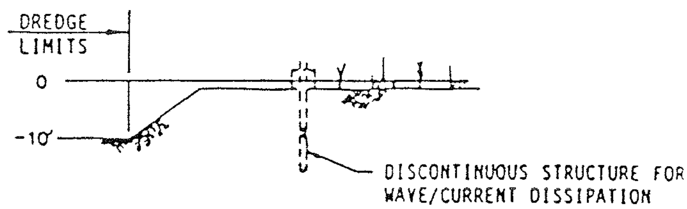
SHEET 5 OF 7

DATE: 9/91
REV. 9/94

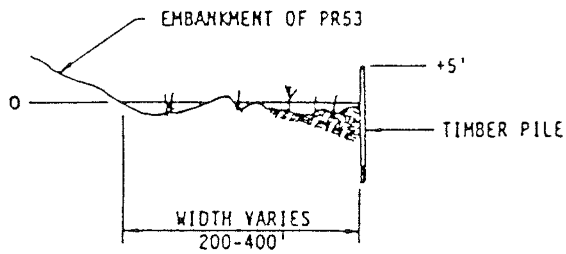
NOTE: ALL CONSTRUCTION IN THIS AREA DELETED FROM ORIGINAL PERMIT.



PLAN VIEW



SECTION A-A



SECTION B-B

JURISDICTIONAL AREA AS DETERMINED BY USCE (SEE SH. 4)

PERIMETER OF PROTECTED WETLANDS

LIMITS OF DREDGING TO PROJECT DEPTH OF -10FT

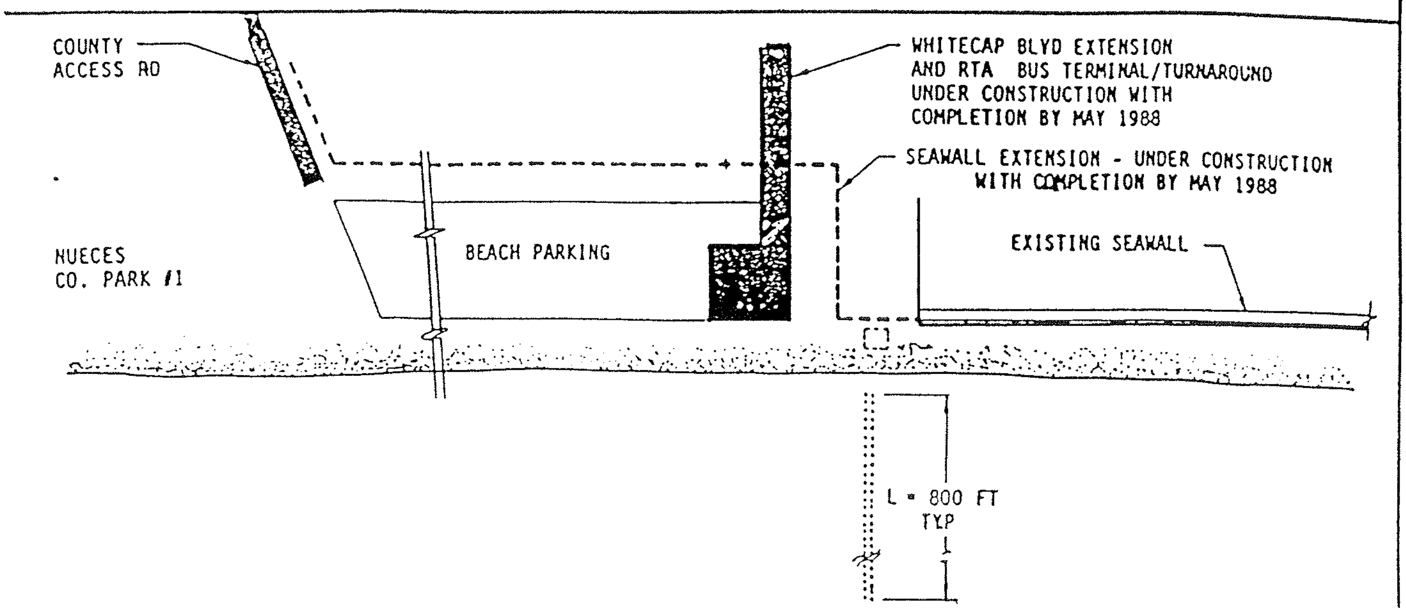
APPROX. SHORELINE AT MEAN TIDE

DISCONTINUOUS STRUCTURE TO PROTECT SHORELINE AND TO ALLOW WATER EXCHANGE INTO WETLANDS. EITHER JETTY-TYPE STONE WITH SPACING BETWEEN OF TIMBER PILES WITH SPACING

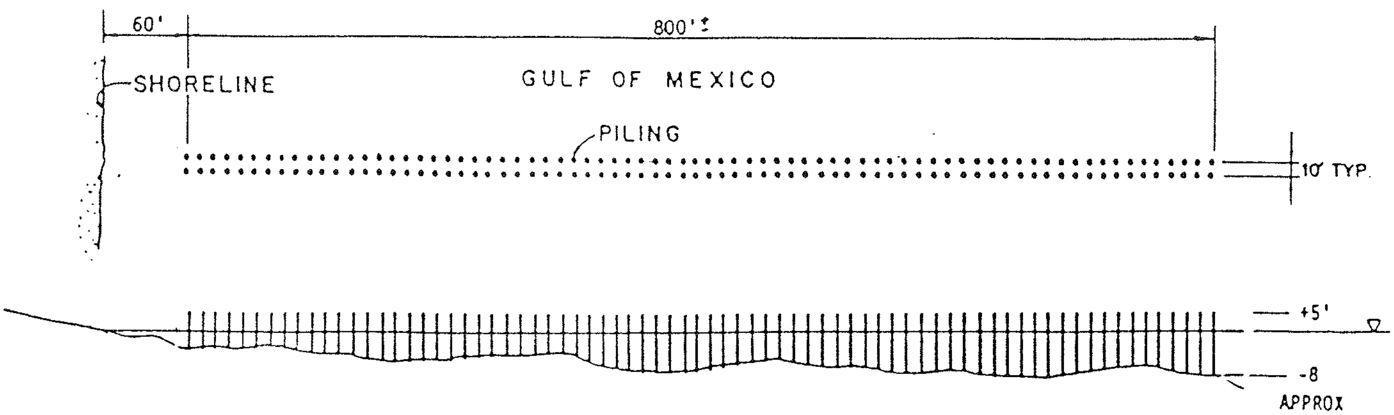
PRESENTLY THIS AREA CONTAINS MOST OF THE VEGETATED WETLANDS FOUND ON THE PROJECT SITE. THIS APPLICATION PROPOSES TO LEAVE EXISTING VEGETATION IN PLACE. WITH THE PROJECT COMPLETION, THE NATURE OF THIS SITE WILL CHANGE DUE TO SALINITY CHANGES AN INCREASE IN TIDAL AMPLITUDE, AND OTHER FACTORS.

SHINER, MOSELEY AND ASSOCIATES
ENGINEERS + CONSULTANTS

PURPOSE: REOPEN PACKERY CHANNEL TO PROVIDE SMALL BOAT ACCESS TO GULF, CONSTRUCT A SMALL BOAT BASIN, TO INCREASE WATER EXCHANGE WITH LAGUNA MADRE THEREBY IMPROVING AQUATIC HABITAT, RELOCATE SURFING PIER DATUM: MEAN SEA LEVEL ADJACENT PROPERTY OWNERS: SEE SHEET 1	PROPOSED: DREDGE CHANNEL, CONSTRUCT BULKHEADS AND JETTIES, INSTALL SAND BYPASS SYSTEM; AND PILINGS COUNTY: NUECES STATE: TEXAS APPLICANT: FISH TRACKERS INC./ REOPEN PACKERY CHANNEL ASSOC. SHEET 6 OF 7 DATE: 9/91 REV. 9/94



POSSIBLE LOCATION



NOTES:

PLAN AND PROFILE

1. PILING TO BE TIMBER
2. APPROPRIATE LIGHTING TO BE PLACED ON OUTER PILING FOR NAVIGATION SAFETY
3. ACTUAL CONFIGURATION MAY VARY TO ALLOW FOR STAGGERED PILING RATHER THAN PARALLEL PAIRS
4. SPECIFIC LOCATION MAY VARY BY UP TO 300 FT. EITHER DIRECTION AND LENGTH COULD BE UP TO 1000 FT.
5. FINAL LOCATION OF SURFING PIER MAY VARY DEPENDING UPON AGREEMENTS REGARDING SITE AVAILABILITY AND ACCESS PROVISIONS.

SHINER, MOSELEY AND ASSOCIATES
ENGINEERS + CONSULTANTS

PURPOSE: REOPEN PACKERY CHANNEL TO PROVIDE SMALL BOAT ACCESS TO GULF, CONSTRUCT A SMALL BOAT BASIN, TO INCREASE WATER EXCHANGE WITH LAGUNA MADRE THEREBY IMPROVING AQUATIC HABITAT, RELOCATE SURFING PIER

DATUM: MEAN SEA LEVEL

ADJACENT PROPERTY OWNERS: SEE SHEET 1

PROPOSED: DREDGE CHANNEL, CONSTRUCT BULKHEADS AND JETTIES, INSTALL SAND BYPASS SYSTEM, AND PILINGS

COUNTY: NUECES STATE: TEXAS

APPLICANT: FISH TRACKERS INC. / REOPEN PACKERY CHANNEL ASSOC.

SHEET 7 OF 7

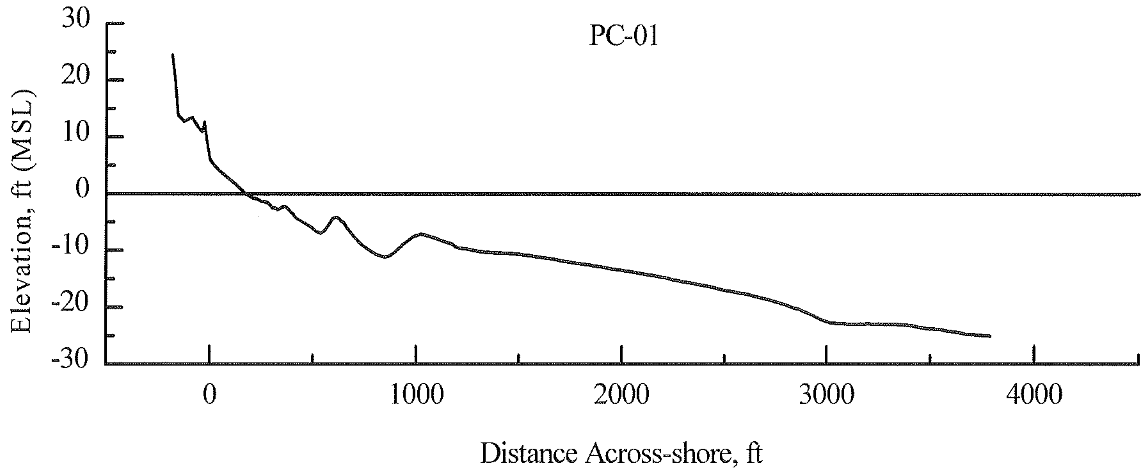
DATE: 9/91
REV. 9/94

Appendix B : Beach Profiles (Historic and New)

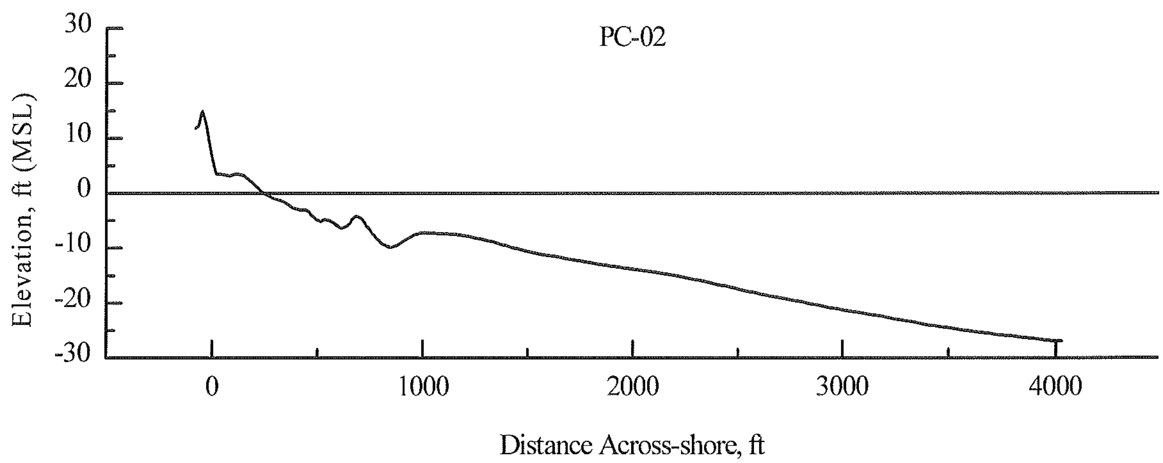
This appendix consists of beach profile data collected April, 1996, at each of the 18 benchmarks established for the shoreline processes and sand management component of the coastal processes assessment associated with re-opening of Packery Channel, Corpus Christi. Specific data points were not identified on the plots because of their high density on each survey line. Locations of the beach profiles are provided in Chapter 2 of this report and in Appendix D. The survey lines were aligned normal to the circumference of a circle of radius 176.4 miles that was fit to the concave trend of the shoreline along north Padre and Mustang Islands. Table B1 lists the azimuths used for each profile line.

This appendix also includes four comparative plots of profiles which re-occupied the positions of a series of beach profiles collected by Watson and Behrens (1976) and Behrens *et al.* (1977) in the early 1970's. Profiles PC13, PC14, PC15, and PC16 were surveyed at locations 2,000 ft north, 150 ft north, 150 ft south, and 2,000 ft south of the Mustang Island Fish Pass so that a quantitative comparison could be made between the historic and present profile shape. Because the original field survey data (including benchmark locations) collected during the 1970 surveys no longer exists, alongshore horizontal positioning of the profile locations was approximated by measuring distance from the jetties. No across-shore control was available. Vertical control was based on use of MSL as a datum (MSL was tied into the 1996 survey using National Oceanic and Atmospheric Administration primary benchmark *5870 A* at Bob Hall Pier). The historic profile data were digitized from original copies of Watson and Behrens (1976) and Behrens *et al.* (1977). In the latter report, profile locations 150 ft north and south, respectively, of the jetties were replaced by locations 200 ft north and south, respectively, of the jetties.

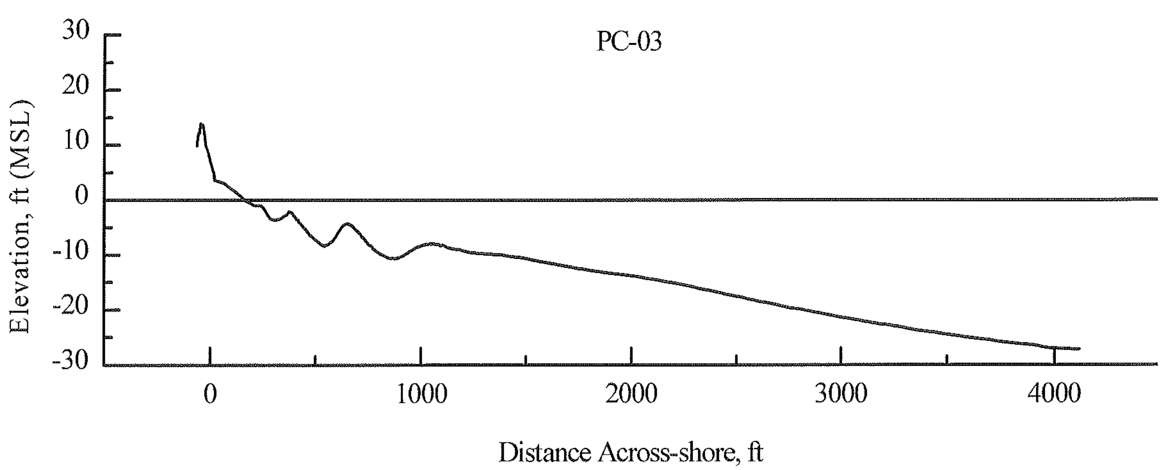
Table B1. Orientation of profile lines.		
Profile	Azimuth from True North	Azimuth from Magnetic North
PC-01	118	112
PC-02	119	113
PC-03	119	113
PC-04	119	113
PC-05	119	113
PC-06	119	113
PC-07	119	113
PC-08	120	114
PC-09	120	114
PC-10	120	114
PC-11	120	114
PC-12	120	114
PC-12A	120	114
PC-12B	121	115
PC-13	122	116
PC-14	122	116
PC-15	122	116
PC-16	122	116



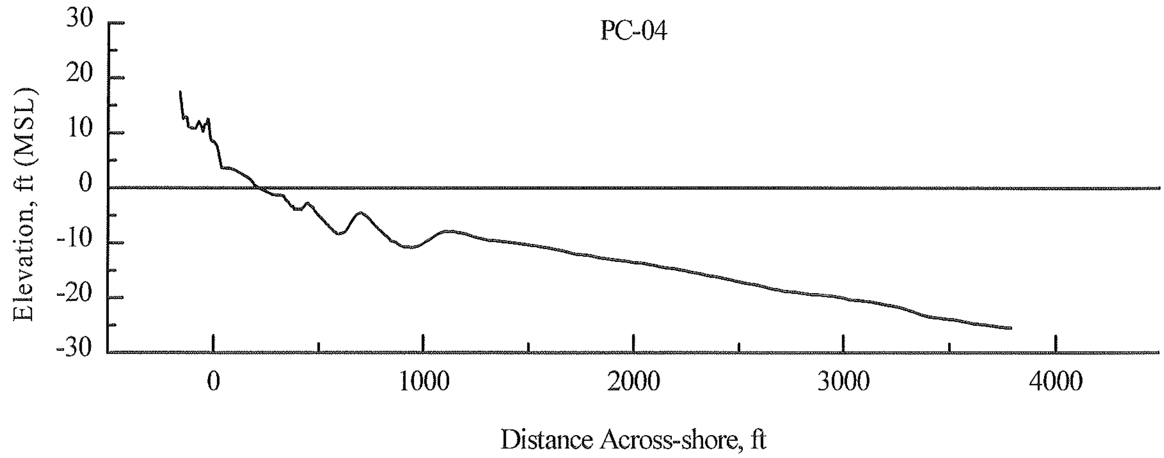
Profile 1: 04/08/96



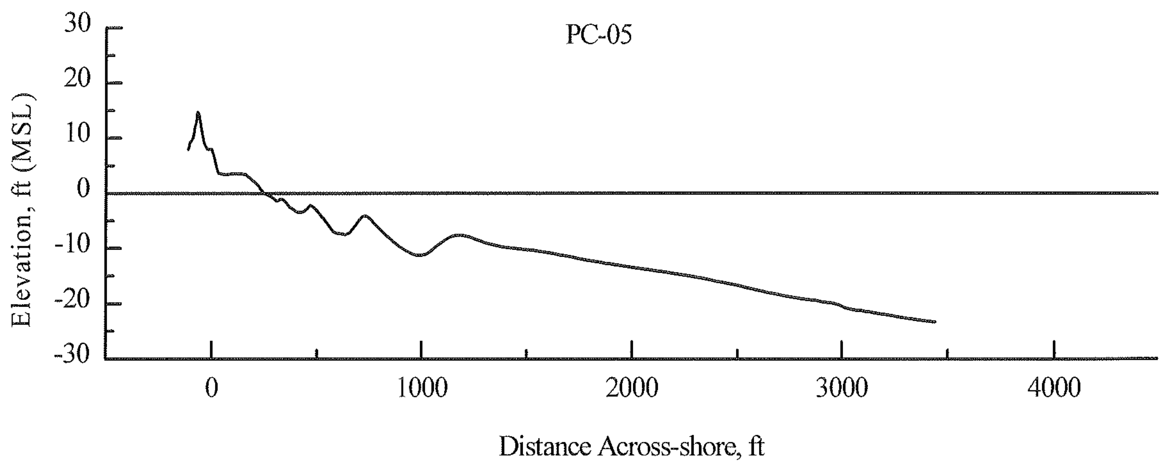
Profile 2: 04/08/96



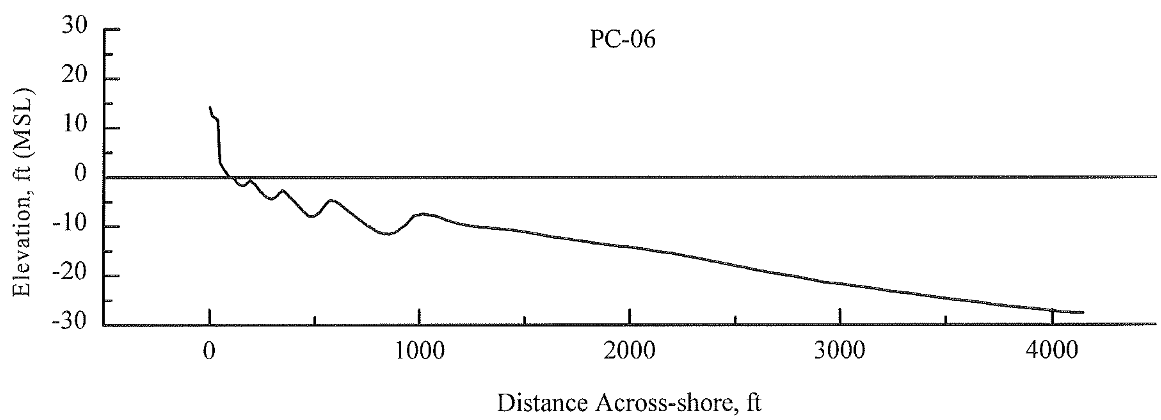
Profile 3: 04/08/96



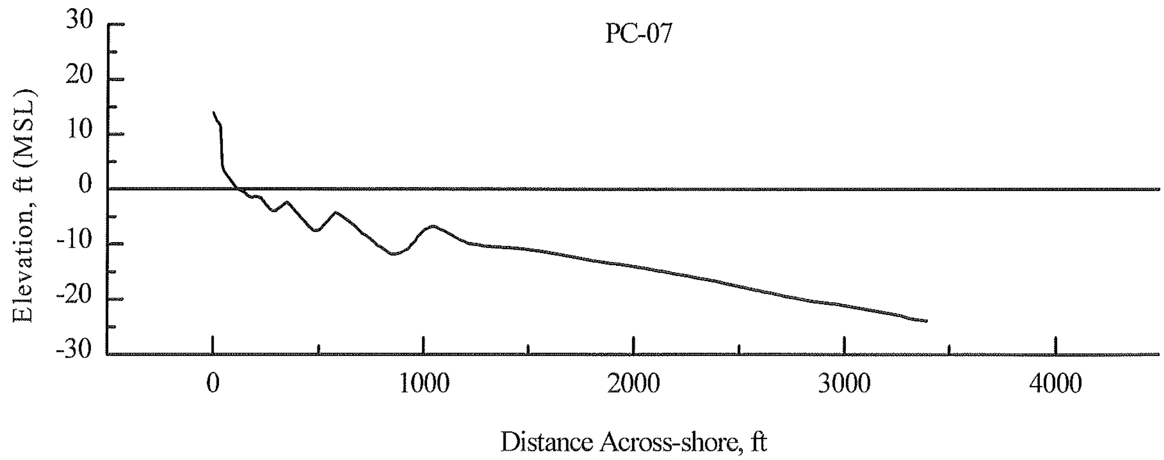
Profile 4: 04/08/96



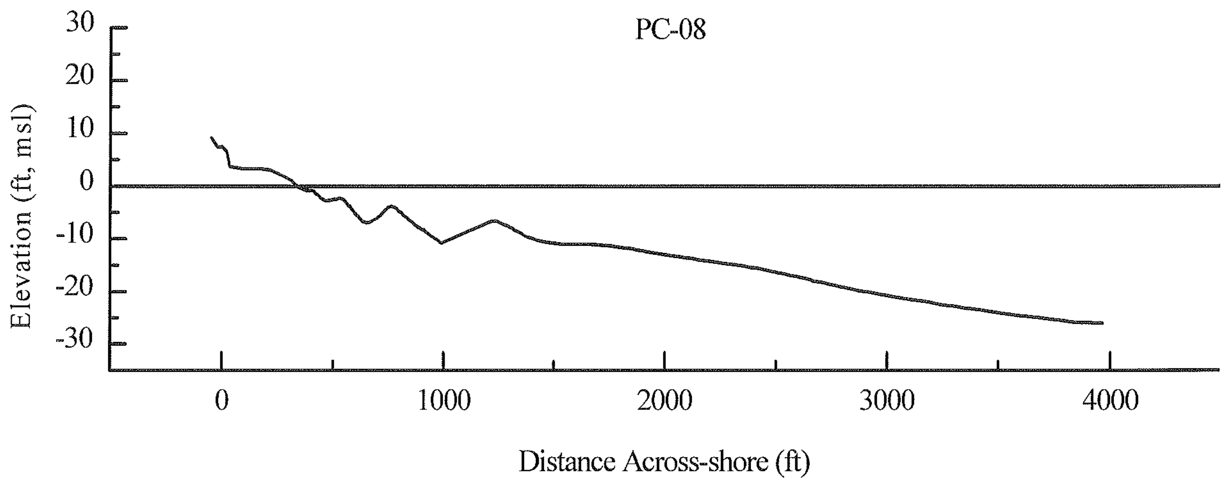
Profile 5: 04/09/96



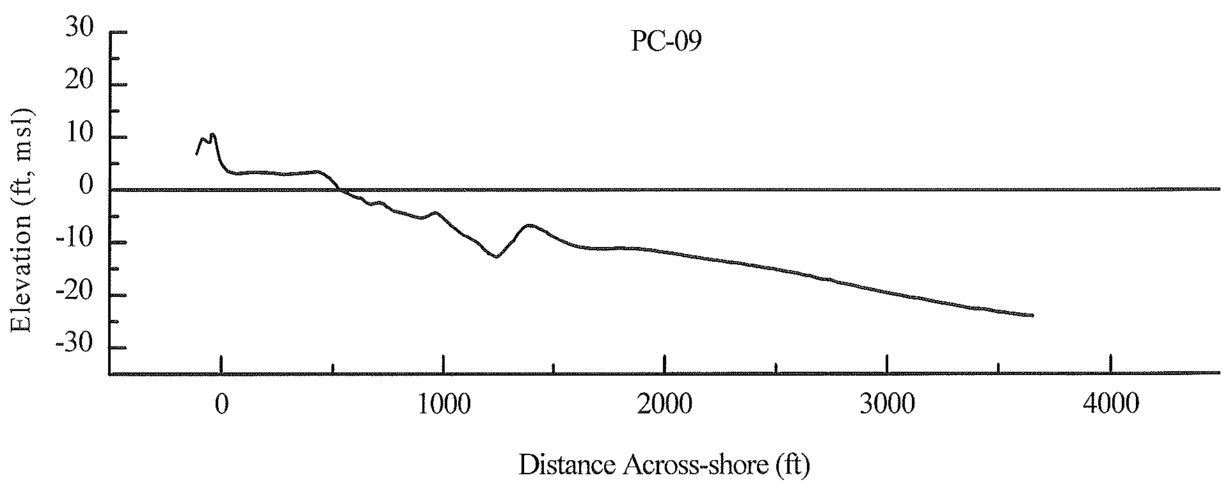
Profile 6: 04/09/96



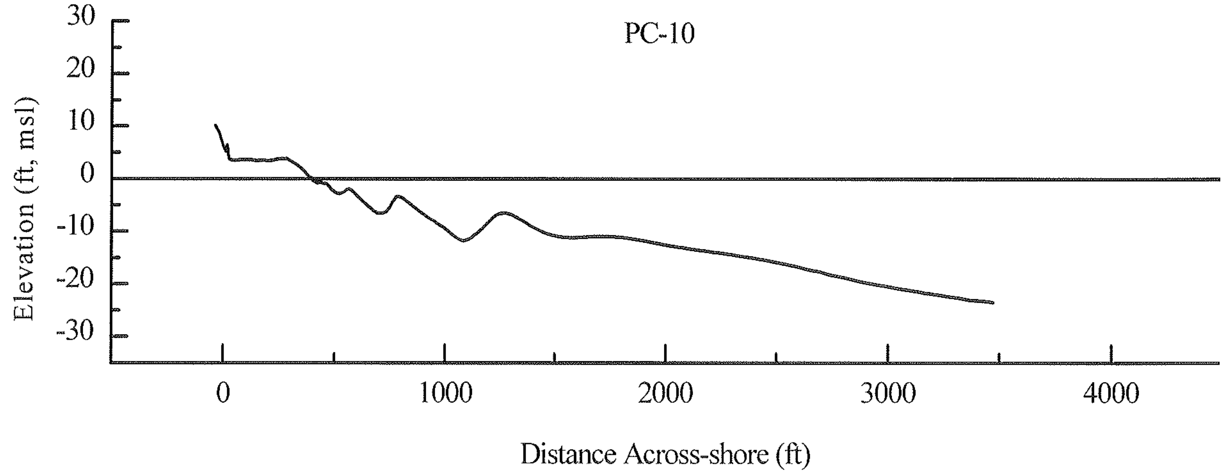
Profile 7: 04/09/96



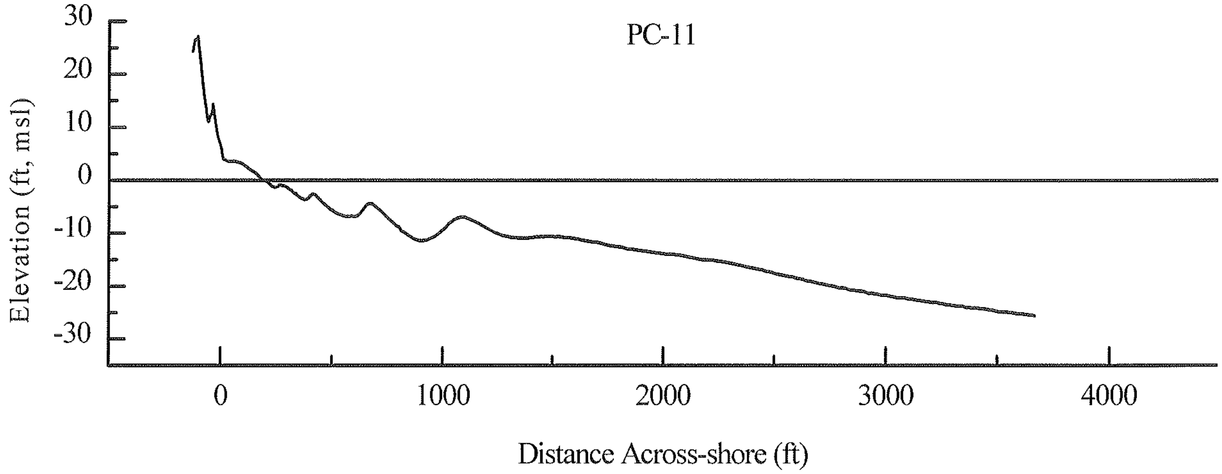
Profile 8: 04/09/96



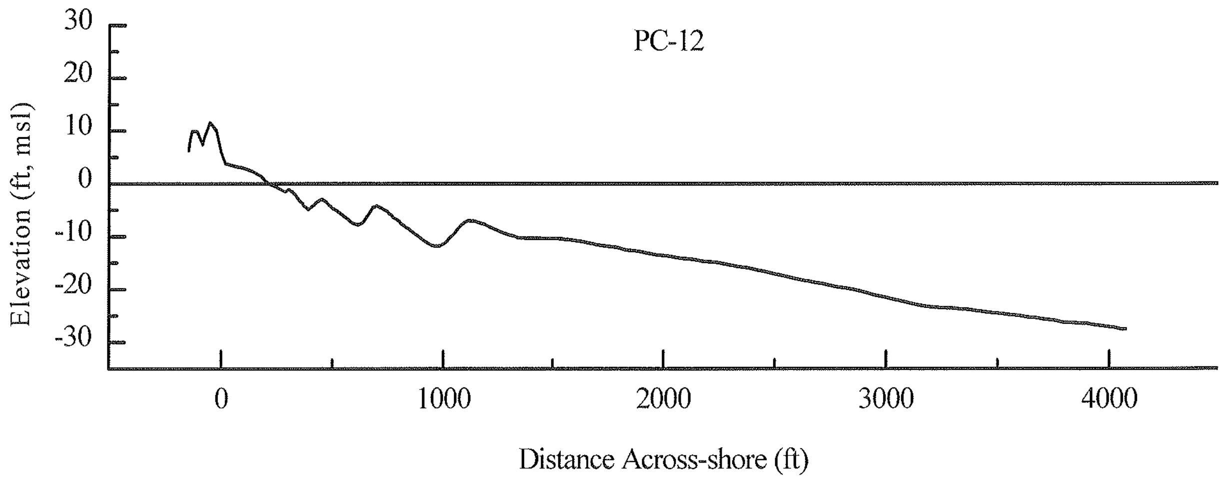
Profile 9: 04/09/96



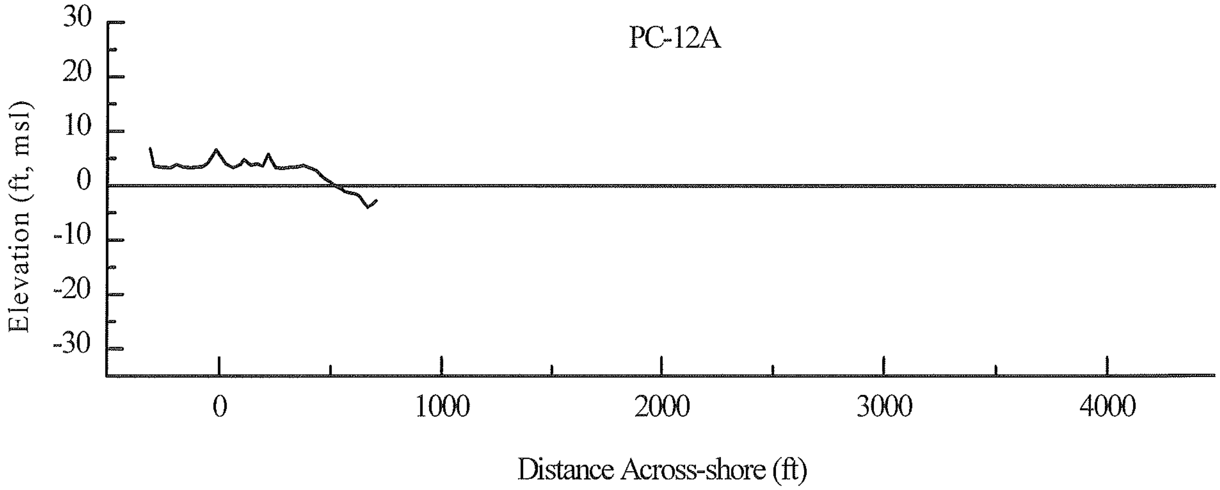
Profile 10: 04/09/96



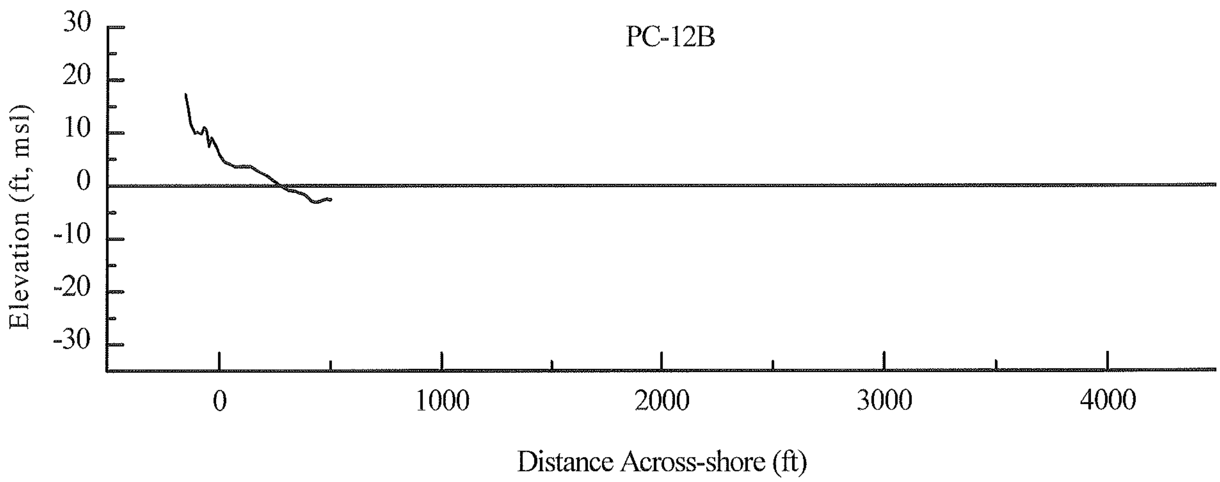
Profile 11: 04/10/96



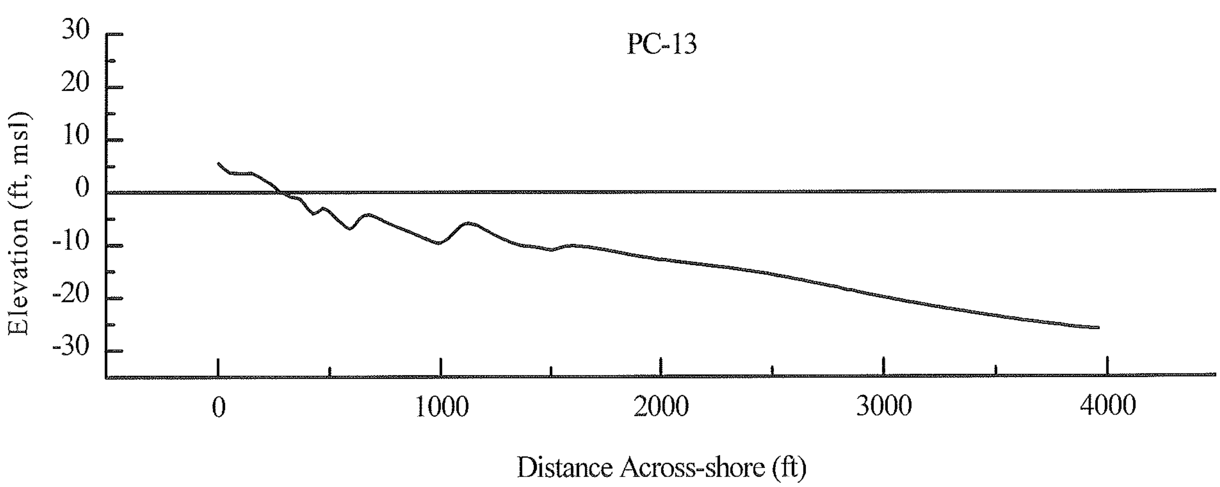
Profile 12: 04/10/96



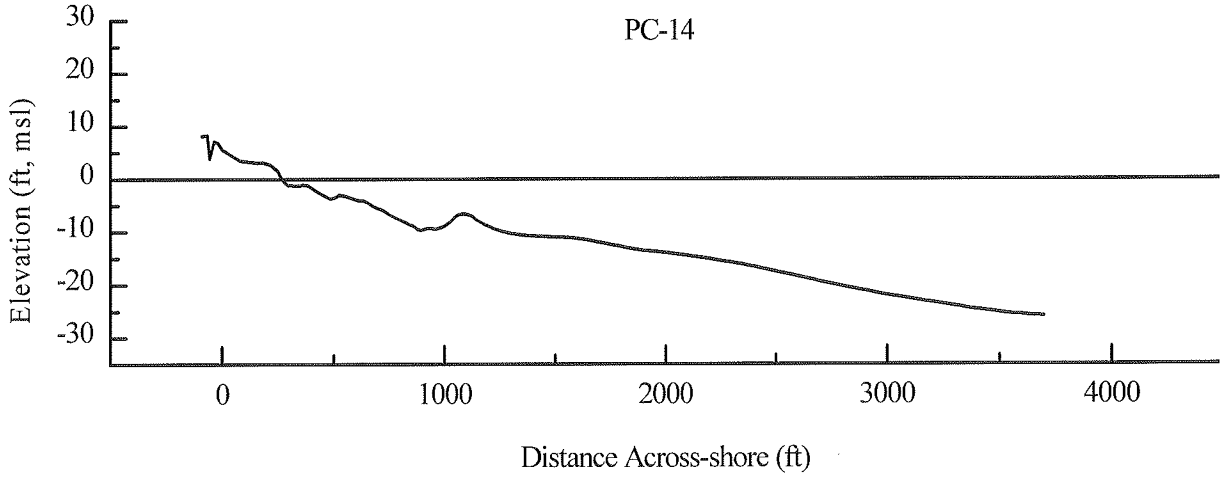
Profile 12A: 04/12/96



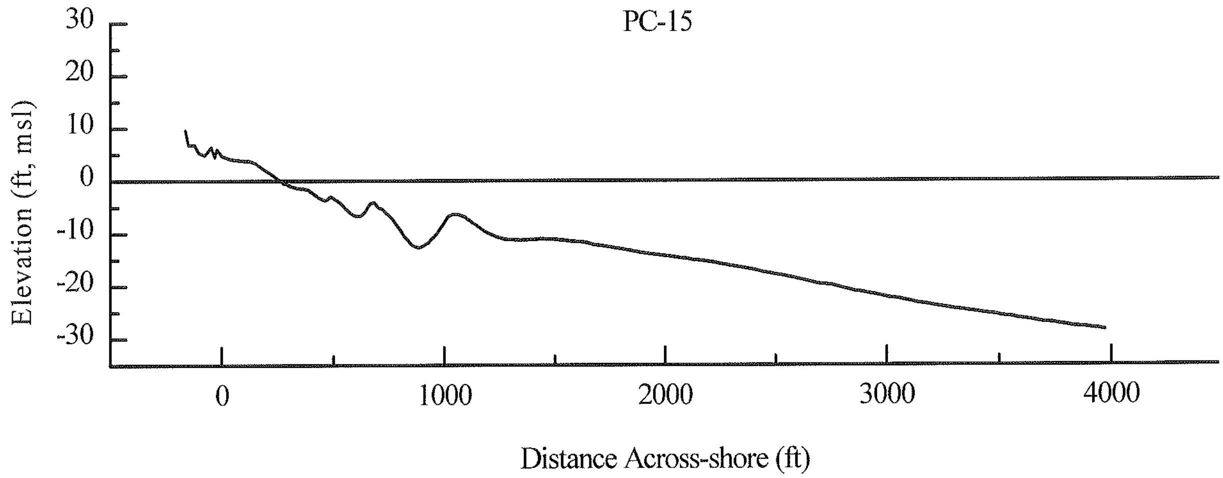
Profile 12B: 04/12/96



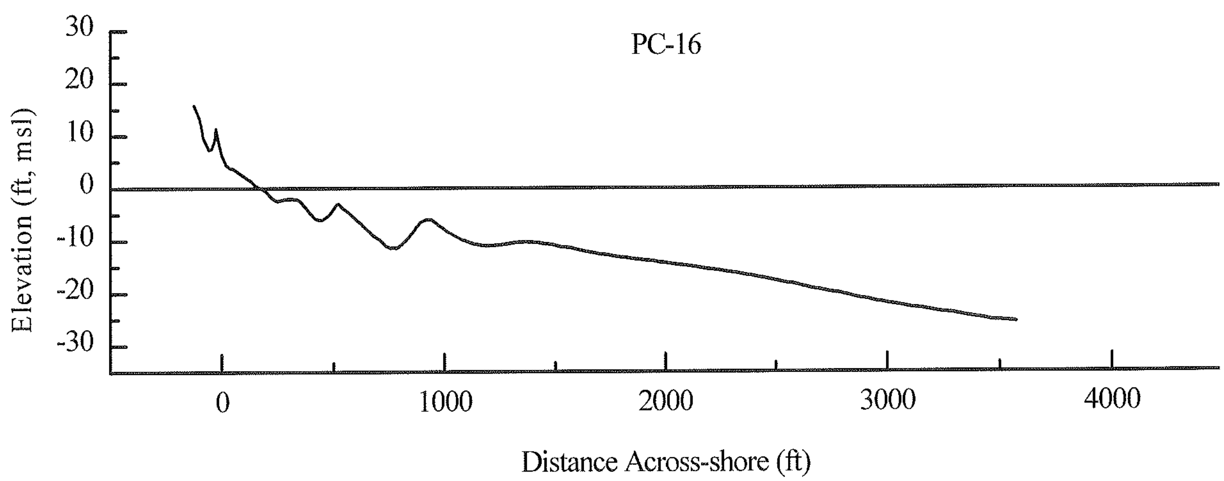
Profile 13: 04/10/96



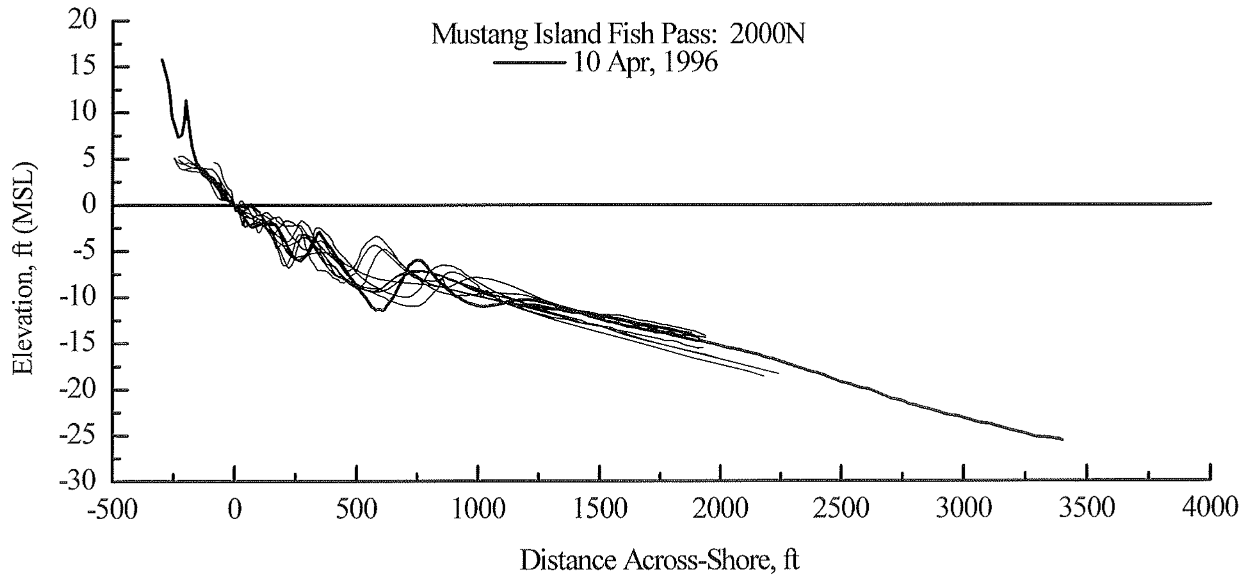
Profile 14: 04/10/96



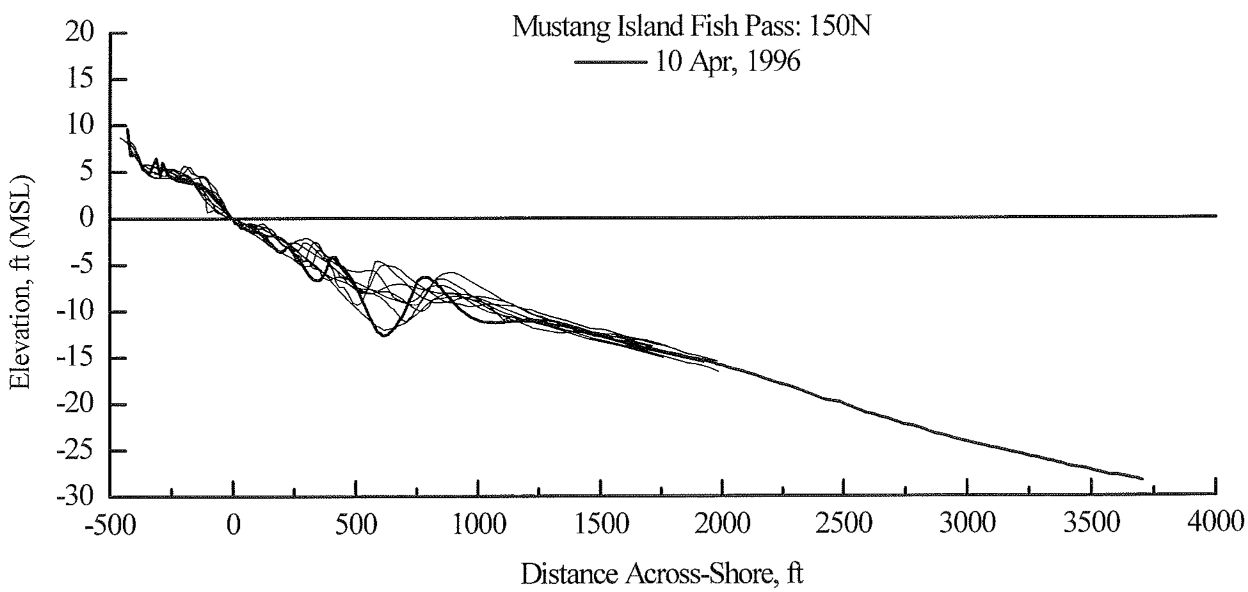
Profile 15: 04/10/96



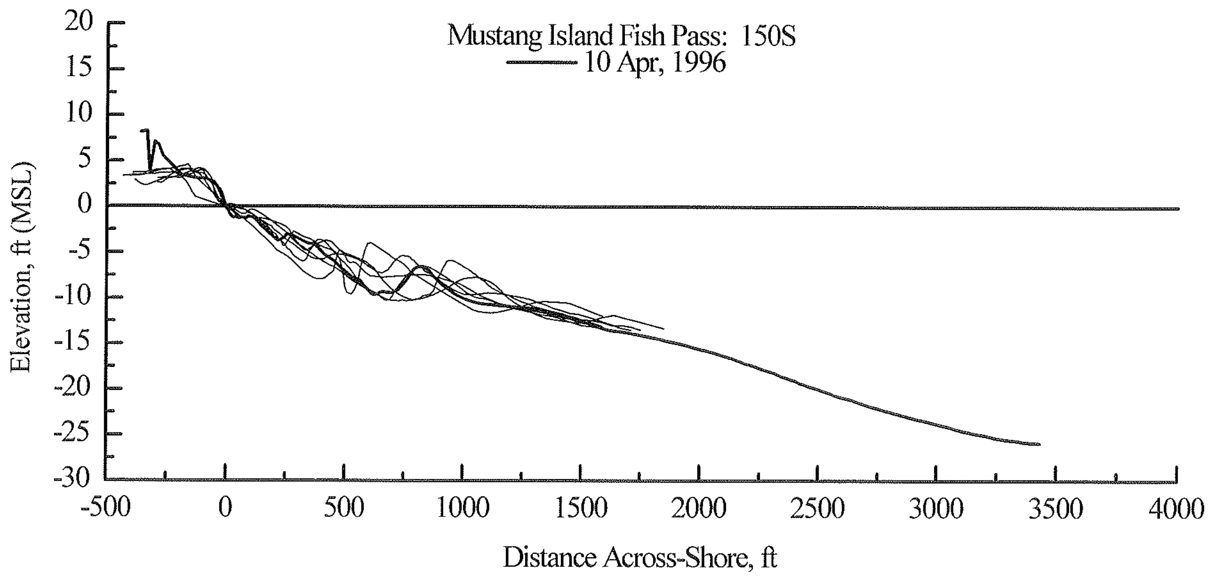
Profile 16: 04/10/96



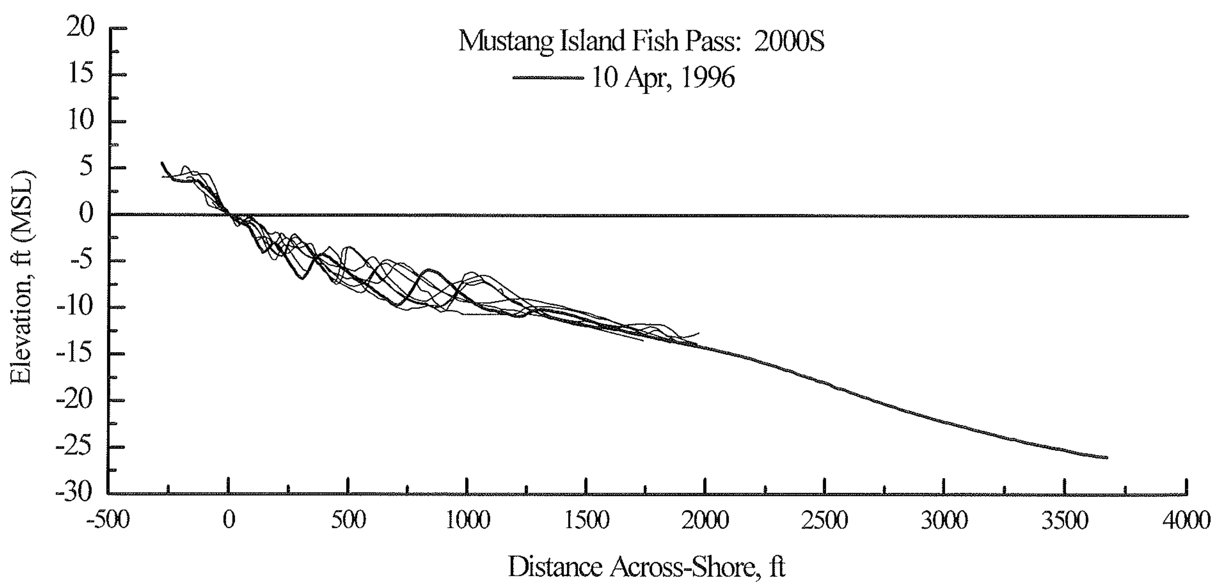
Profile 2000N: Mustang Island Fish Pass (profiles shifted to common shoreline). Includes data collected on the following dates:
From Behrens *et al.* (1977): 10/9/72, 11/9/72, 12/21/72, 4/4/73, 6/27/73;
From Watson and Behrens (1976): 6/28/73, 6/12/74, 12/5/74, 3/19/75.



Profile 150N: Mustang Island Fish Pass (profiles shifted to common shoreline). Includes data collected on the following dates:
From Behrens *et al.* (1977): 10/9/72, 11/9/72, 12/21/72, 4/4/73 6/28/73;
From Watson and Behrens (1976): 6/28/73, 6/12/74, 12/5/74, 3/19/75.



Profile 150S: Mustang Island Fish Pass (profiles shifted to common shoreline). Includes data collected on the following dates:
From Behrens *et al.* (1977): 10/9/72, 1/16/73, 3/17/73;
From Watson and Behrens (1976): 6/27/73, 6/12/74, 3/19/75.



Profile 2000S: Mustang Island Fish Pass (profiles shifted to common shoreline). Includes data collected on the following dates:
From Behrens *et al.* (1977): 10/10/72, 1/16/73, 3/17/73, 6/27/73;
From Watson and Behrens (1976): 6/28/73, 6/12/74, 3/19/75.

Appendix C : Sediment Grain-Size Data

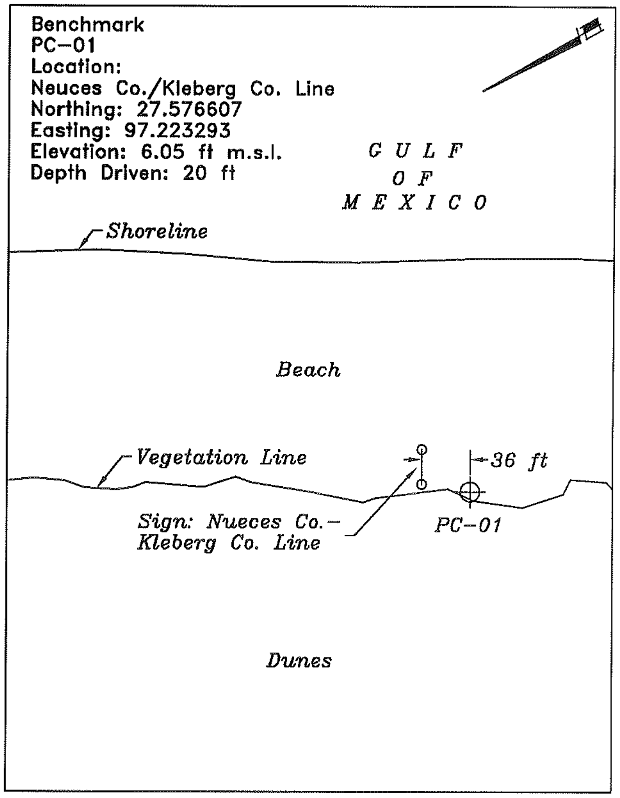
This Appendix contains quantitative descriptions of sediment grain-size obtained from samples taken at the study site by Blucher Institute personnel. Grain size was determined by laser light diffraction principles.

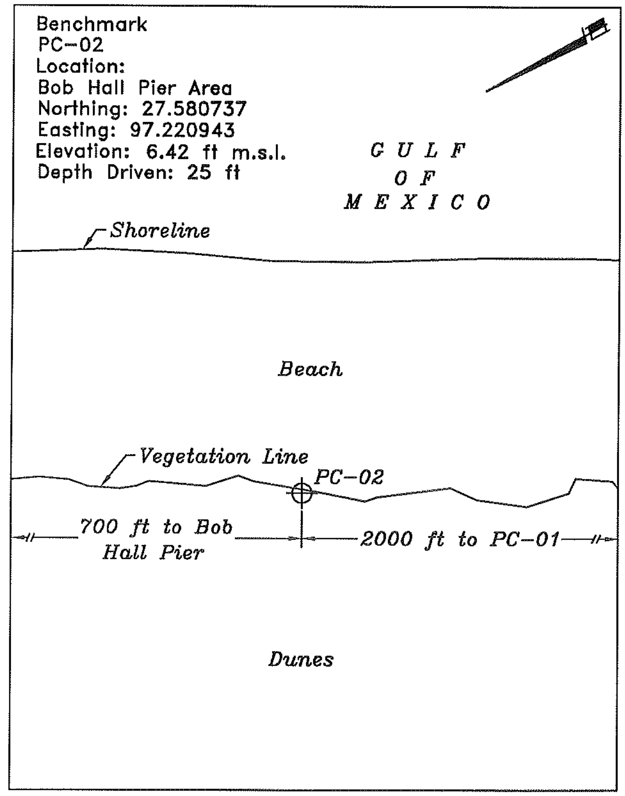
Table C1 contains grain-size analysis results for the samples taken during the beach profile survey conducted during April 8-10, 1996. The median grain size, mode, and standard deviation in the distribution are given. The standard deviation indicates the variety in grain size from coarse to fine. The median grain size along the north Padre and Mustang Island beaches varies little, typically lying between 0.17 and 0.20 mm, as determined by diffraction analysis. The results found for median grain size are in accord with those reported by Mason and Folk (1958), Gage (1970), Garner (1967), and others.

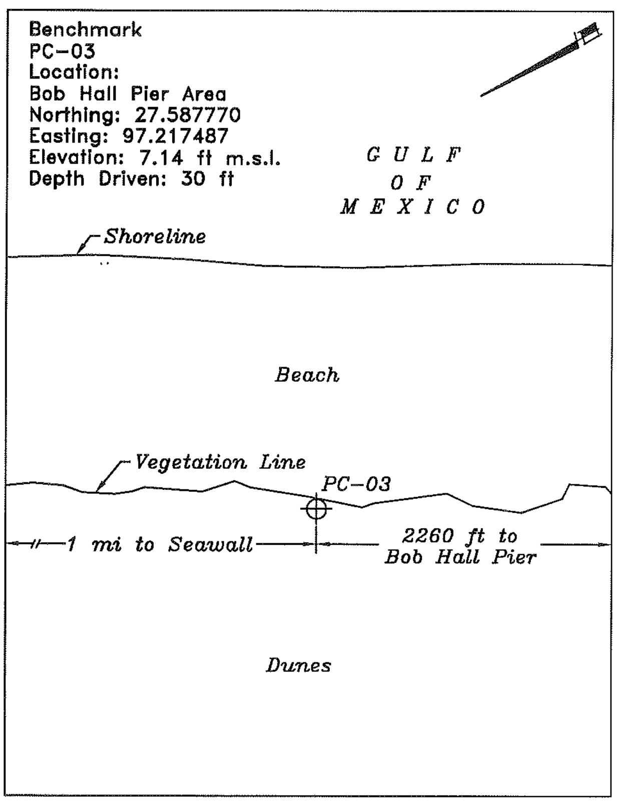
Table C1: Median, mode, and standard deviation of grain sizes in the sediment samples collected during the beach profile survey. Units: mm						
Profile Line No.	Foot of Dune	Mid-Berm	Shoreline	3-ft Depth	12-ft Depth	24-ft Depth
PC-01	0.19 0.19 0.07	0.18 0.18 0.06	0.18 0.18 0.07	-------------------	0.13 0.13 0.07	0.12 0.12 0.06
PC-02	0.21 0.20 0.06	0.20 0.20 0.08	0.27 0.25 0.14	0.18 0.19 0.06	0.12 0.13 0.08	-------------------
PC-03	0.19 0.20 0.06	0.19 0.19 0.08	0.20 0.20 0.08	0.22 0.19 0.12	0.15 0.13 0.13	0.13 0.13 0.06
PC-04	0.18 0.18. 0.07	0.18 0.18 0.07	0.19 0.19 0.08	0.15 0.15 0.07	0.13 0.13 0.07	0.12 0.12 0.06
PC-05	0.17 0.17 0.06	0.17 0.16 0.07	-------------------	0.27 0.27 0.13	0.14 0.12 0.13	0.12 0.12 0.05
PC-06	0.19 0.20 0.07	0.20 0.20 0.08	0.19 0.19 0.08	0.16 0.16 0.07	-------------------	0.19 0.21 0.11
PC-07	0.21 0.21 0.08	0.21 0.22 0.09	0.18 0.18 0.08	0.15 0.15 0.06	0.13 0.13 0.06	0.13 0.13 0.06
PC-08	0.19 0.18 0.07	0.18 0.19 0.08	0.20 0.20 0.08	0.15 0.14 0.08	0.13 0.13 0.06	0.12 0.12 0.06
PC-09	0.19 0.19 0.06	0.18 0.18 0.07	0.18 0.18 0.08	0.18 0.17 0.08	0.13 0.13 0.05	0.11 0.11 0.05
PC-10	0.19 0.19 0.08	0.20 0.20 0.09	0.18 0.18 0.07	0.18 0.18 0.08	0.13 0.13 0.05	0.11 0.17 0.13
PC-11	0.17 0.17 0.06	0.18 0.18 0.09	0.20 0.20 0.09	0.19 0.18 0.10	0.13 0.13 0.07	0.12 0.11 0.04
PC-12	0.18 0.17 0.07	0.17 0.17 0.06	0.19 0.18 0.09	0.19 0.19 0.08	0.12 0.12 0.06	0.13 0.10 0.09
PC-12A	0.17 0.16 0.06	0.17 0.17 0.07	0.18 0.17 0.07	0.21 0.19 0.14	-------------------	-------------------
PC-12B	0.17 0.16 0.05	0.17 0.17 0.07	0.18 0.18 0.08	0.16 0.16 0.06	-------------------	-------------------
PC-13	0.16 0.16 0.06	0.18 0.18 0.07	0.27 0.25 0.13	0.21 0.20 0.10	0.13 0.13 0.06	0.26 0.27 0.14
PC-14	0.17 0.17 0.07	0.20 0.20 0.08	0.18 0.18 0.07	0.14 0.14 0.07	-------------------	-------------------
PC-15	0.16 0.17 0.06	0.18 0.18 0.07	0.20 0.20 0.10	0.19 0.19 0.07	0.14 0.14 0.07	0.13 0.13 0.05
PC-16	-------------------	0.17 0.17 0.07	0.20 0.20 0.09	0.21 0.18 0.15	0.12 0.12 0.06	0.15 0.15 0.07

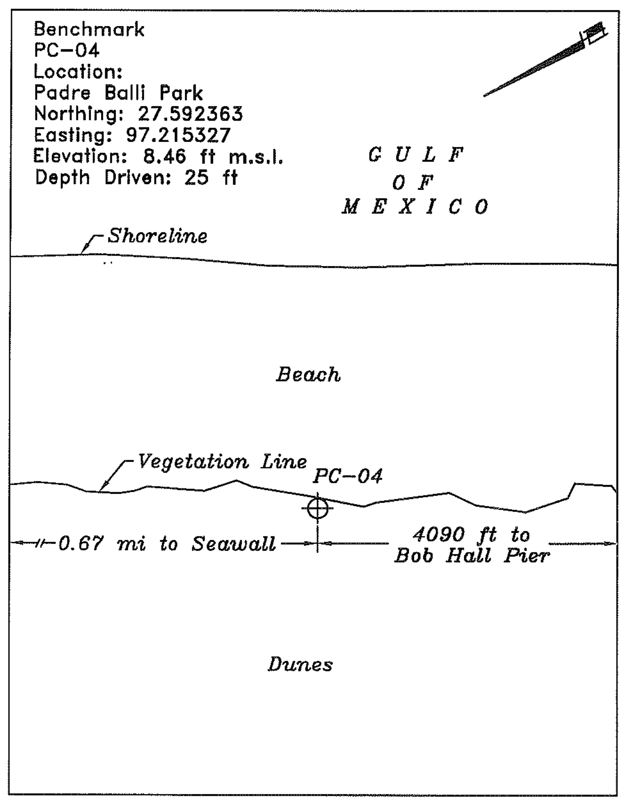
Appendix D : Benchmark Location Maps

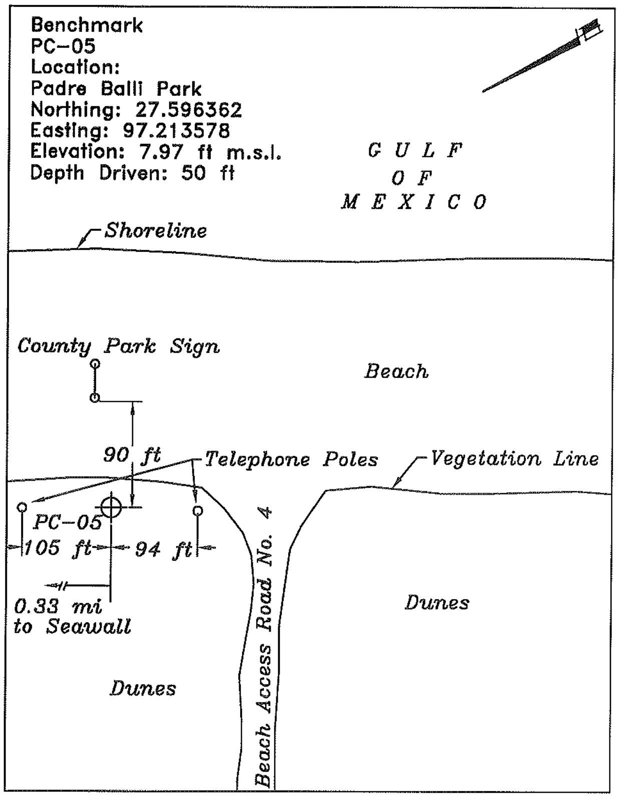
This appendix provides location diagrams for each of the 18 survey benchmarks established for the beach profile survey associated with re-opening of Packery Channel. Each benchmark was installed with a capped 3-ft section of 4-inch diameter PVC pipe encased in a concrete kickblock.

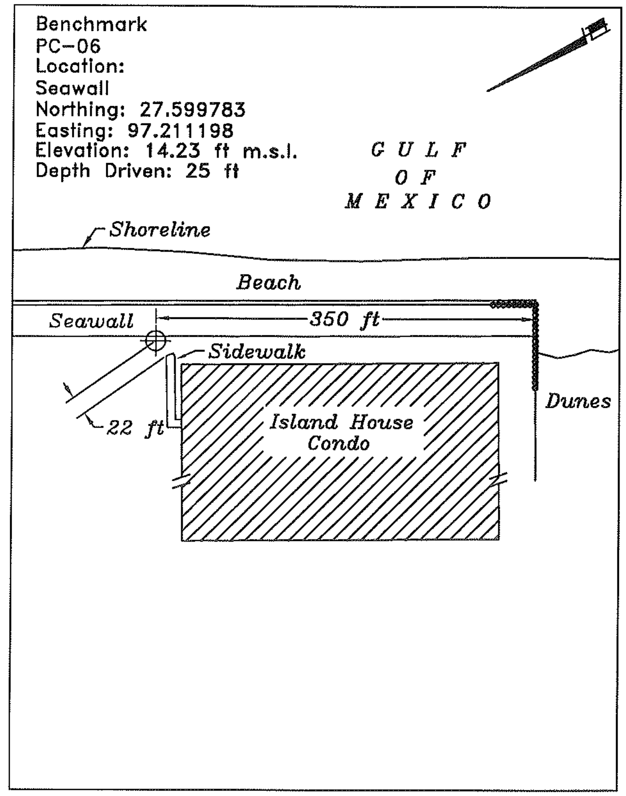

Benchmark
PC-01
Location:
Neuces Co./Kleberg Co. Line
Northing: 27.576607
Easting: 97.223293
Elevation: 6.05 ft m.s.l.
Depth Driven: 20 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
36 ft
Sign: Nueces Co.–
Kleberg Co. Line
PC–01
Dunes

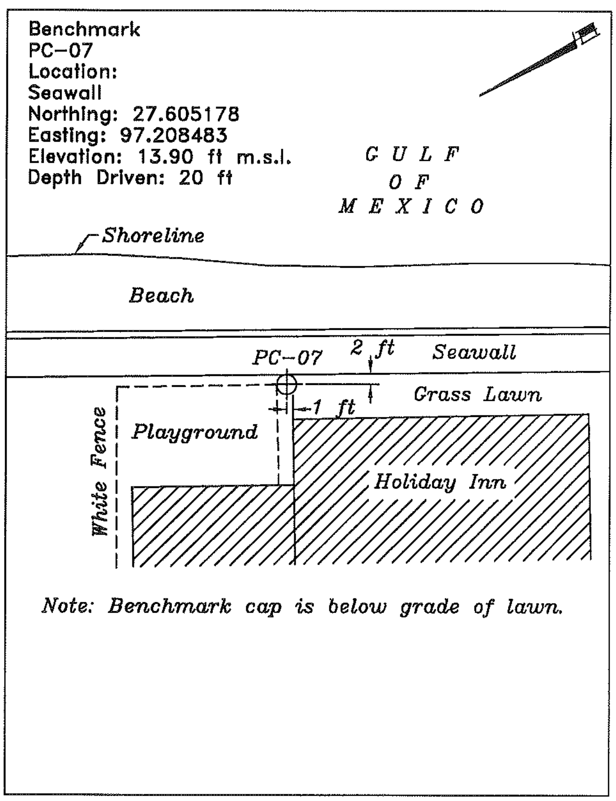

Benchmark
PC-02
Location:
Bob Hall Pier Area
Northing: 27.580737
Easting: 97.220943
Elevation: 6.42 ft m.s.l.
Depth Driven: 25 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
PC–02
700 ft to Bob
Hall Pier
2000 ft to PC–01
Dunes

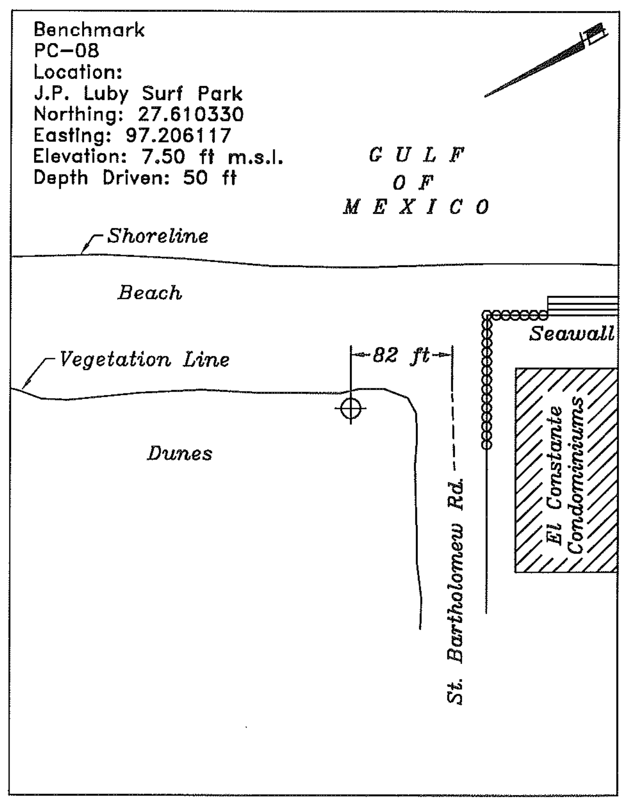

Benchmark
PC-03
Location:
Bob Hall Pier Area
Northing: 27.587770
Easting: 97.217487
Elevation: 7.14 ft m.s.l.
Depth Driven: 30 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
PC–03
1 mi to Seawall
2260 ft to
Bob Hall Pier
Dunes

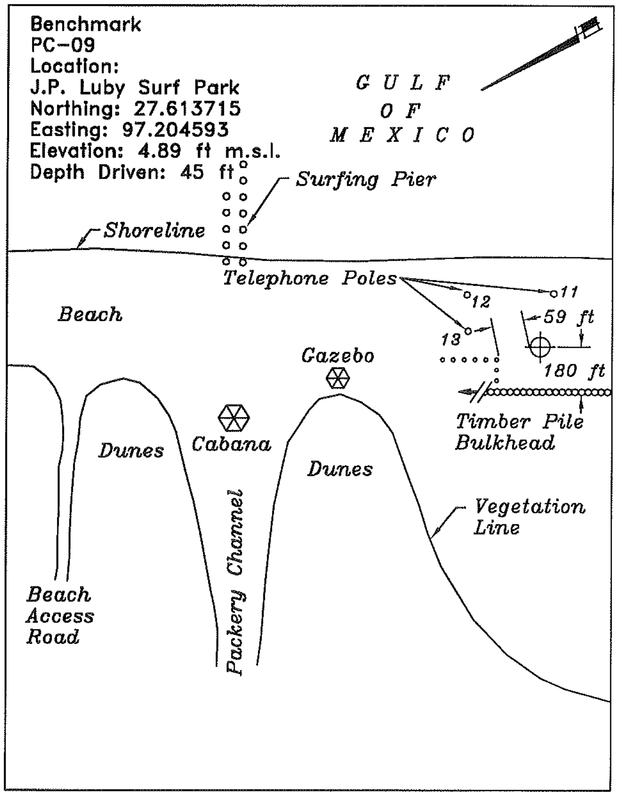

Benchmark
PC-04
Location:
Padre Balli Park
Northing: 27.592363
Easting: 97.215327
Elevation: 8.46 ft m.s.l.
Depth Driven: 25 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
PC–04
0.67 mi to Seawall
4090 ft to
Bob Hall Pier
Dunes

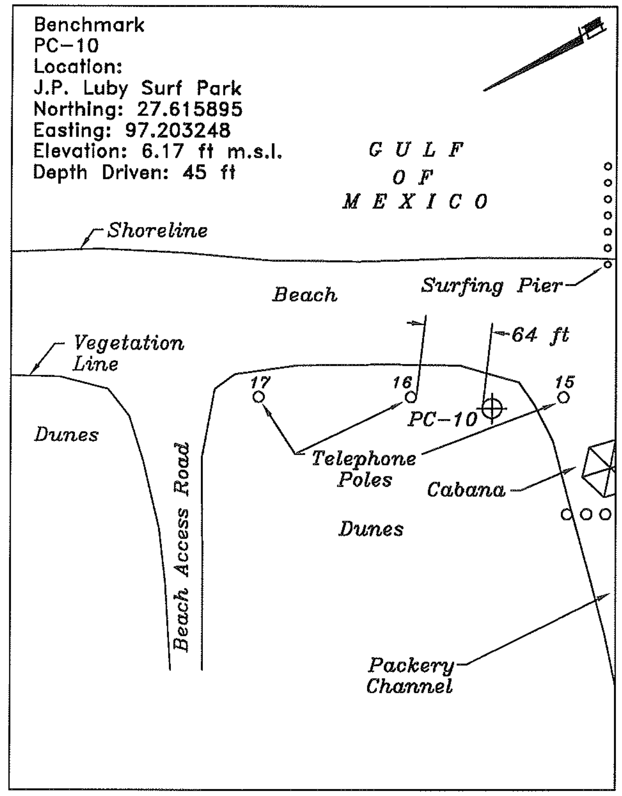

Benchmark
PC-05
Location:
Padre Balli Park
Northing: 27.596362
Easting: 97.213578
Elevation: 7.97 ft m.s.l.
Depth Driven: 50 ft
GULF OF MEXICO
Shoreline
County Park Sign
Beach
90 ft
Telephone Poles
Vegetation Line
PC-05
105 ft
94 ft
0.33 mi to Seawall
Beach Access Road No. 4
Dunes
Dunes

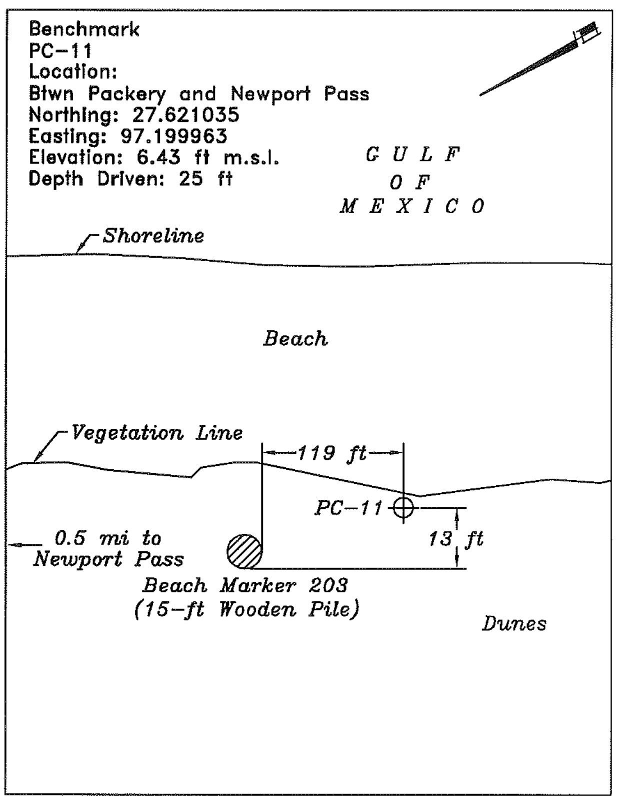

Benchmark
PC-06
Location:
Seawall
Northing: 27.599783
Easting: 97.211198
Elevation: 14.23 ft m.s.l.
Depth Driven: 25 ft
GULF OF MEXICO
Shoreline
Beach
Seawall
350 ft
Sidewalk
22 ft
Island House Condo
Dunes

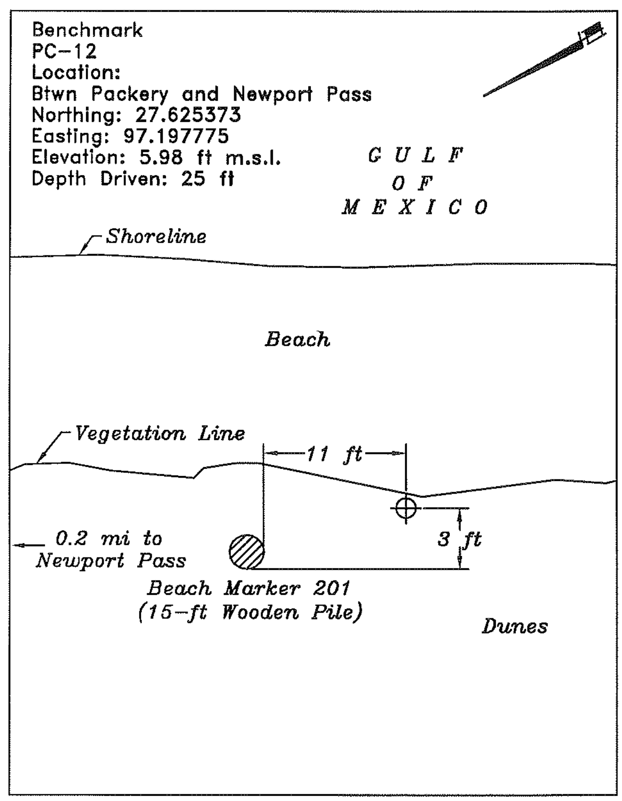

Benchmark
PC-07
Location:
Seawall
Northing: 27.605178
Easting: 97.208483
Elevation: 13.90 ft m.s.l.
Depth Driven: 20 ft
GULF OF MEXICO
Shoreline
Beach
PC-07
2 ft
Seawall
Grass Lawn
1 ft
Playground
White Fence
Holiday Inn
Note: Benchmark cap is below grade of lawn.

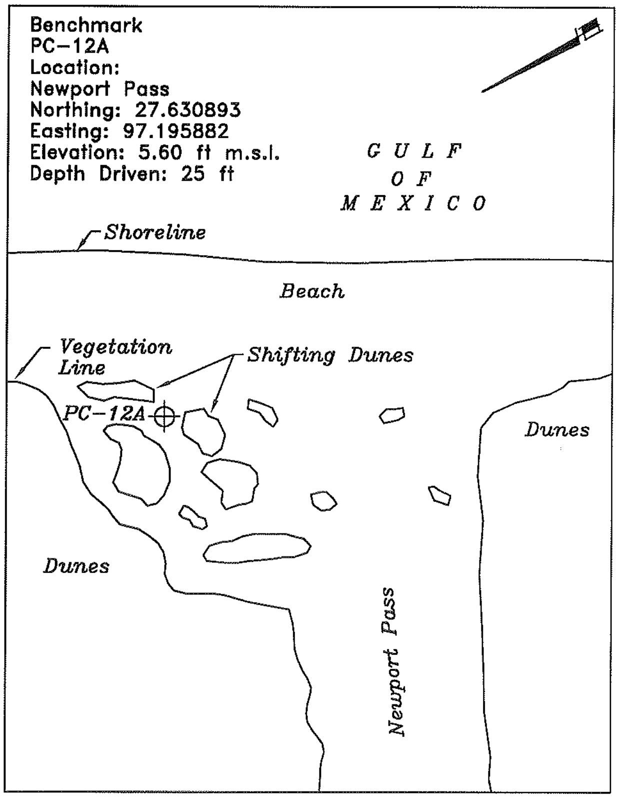

Benchmark
PC-08
Location:
J.P. Luby Surf Park
Northing: 27.610330
Easting: 97.206117
Elevation: 7.50 ft m.s.l.
Depth Driven: 50 ft
GULF OF MEXICO
Shoreline
Beach
Seawall
Vegetation Line
82 ft
Dunes
St. Bartholomew Rd.
El Constante Condominiums

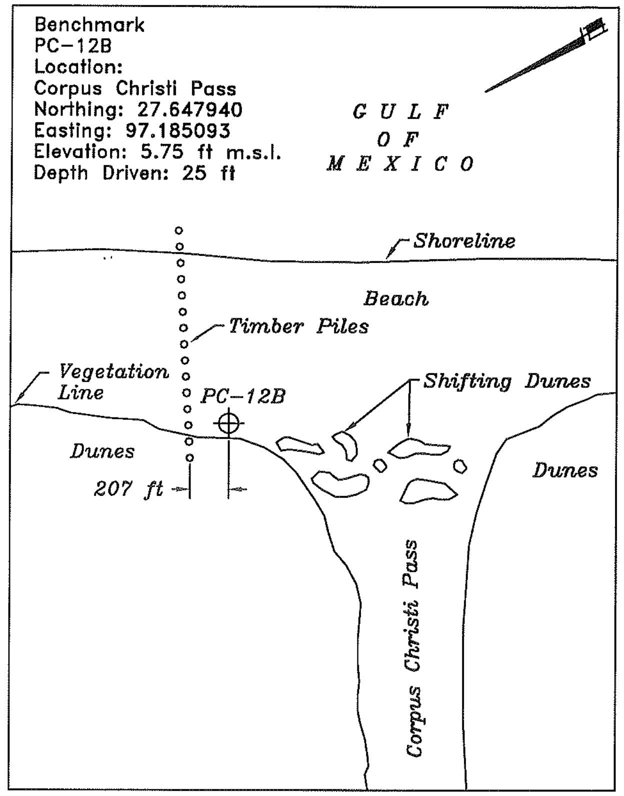

Benchmark
PC-09
Location:
J.P. Luby Surf Park
Northing: 27.613715
Easting: 97.204593
Elevation: 4.89 ft m.s.l.
Depth Driven: 45 ft
G U L F
O F
M E X I C O
Surfing Pier
Shoreline
Telephone Poles
11
12
13
59 ft
180 ft
Beach
Gazebo
Timber Pile
Bulkhead
Dunes
Cabana
Dunes
Vegetation
Line
Packery Channel
Beach
Access
Road

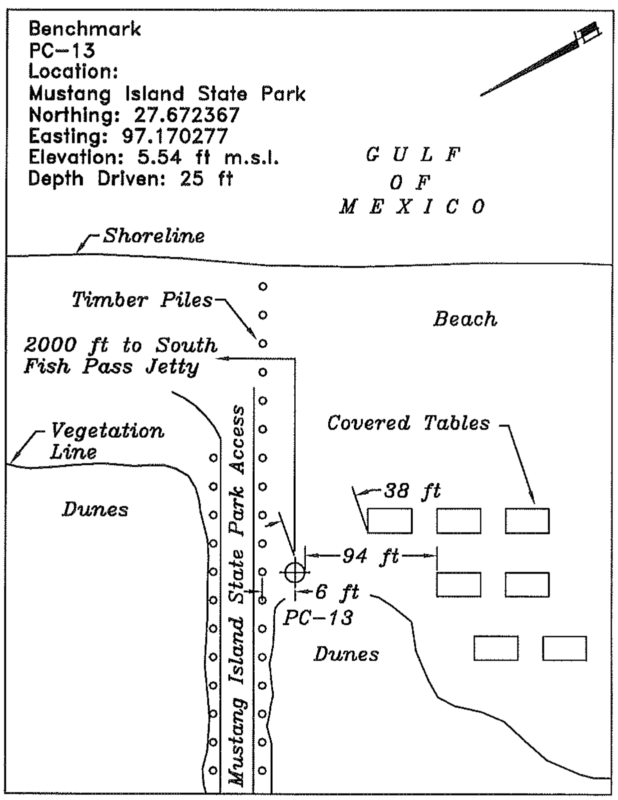

Benchmark
PC-10
Location:
J.P. Luby Surf Park
Northing: 27.615895
Easting: 97.203248
Elevation: 6.17 ft m.s.l.
Depth Driven: 45 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Surfing Pier
Vegetation
Line
64 ft
17
16
15
PC-10
Dunes
Beach Access Road
Telephone
Poles
Cabana
Dunes
Packery
Channel

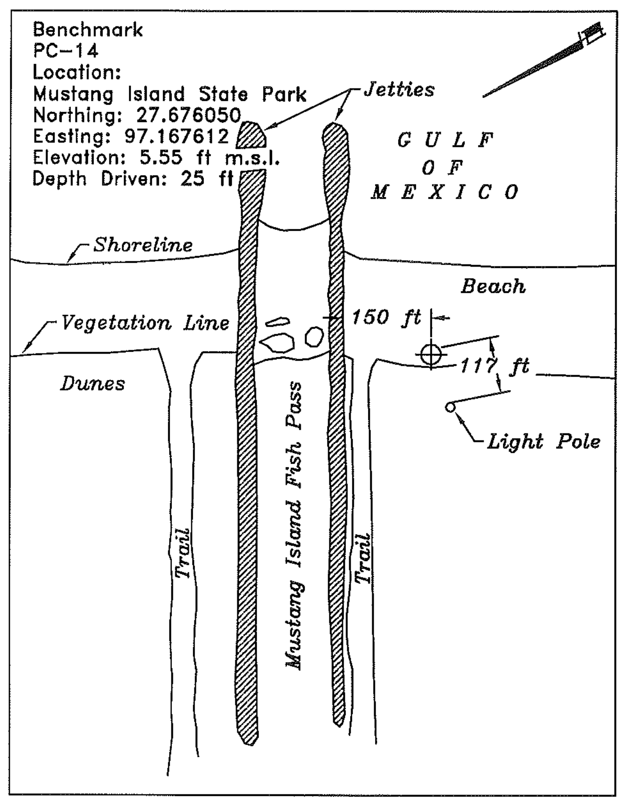

Benchmark
PC-11
Location:
Btwn Packery and Newport Pass
Northing: 27.621035
Easting: 97.199963
Elevation: 6.43 ft m.s.l.
Depth Driven: 25 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
119 ft
PC-11
0.5 mi to
Newport Pass
13 ft
Beach Marker 203
(15-ft Wooden Pile)
Dunes

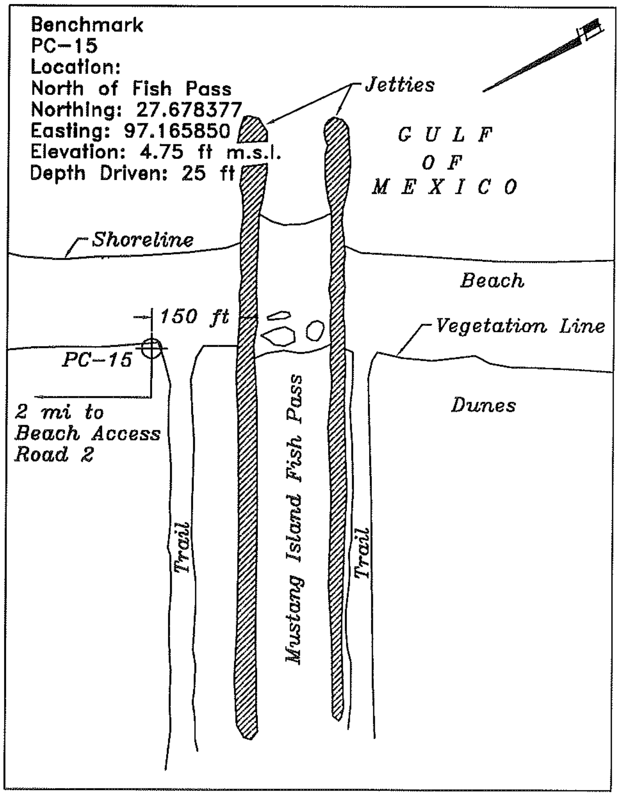

Benchmark
PC-12
Location:
Btwn Packery and Newport Pass
Northing: 27.625373
Easting: 97.197775
Elevation: 5.98 ft m.s.l.
Depth Driven: 25 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
11 ft
0.2 mi to
Newport Pass
3 ft
Beach Marker 201
(15-ft Wooden Pile)
Dunes

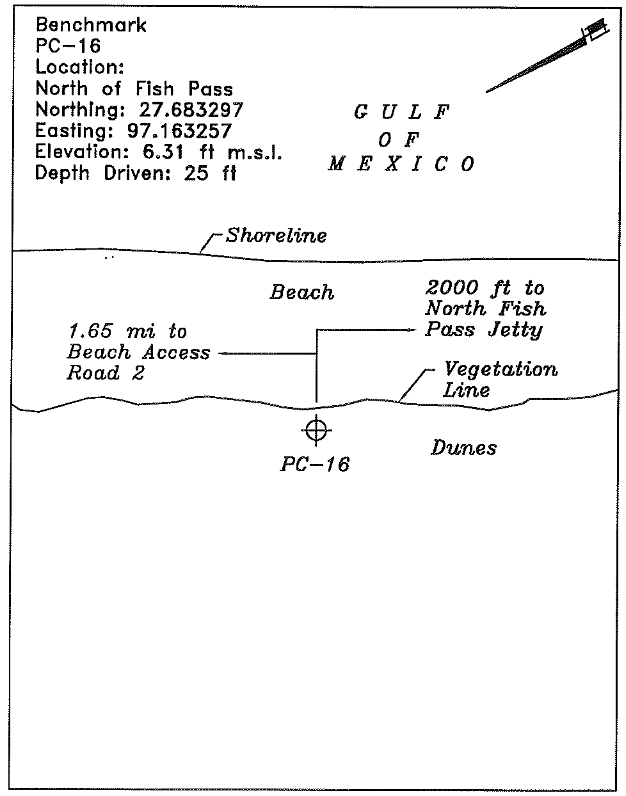

Benchmark
PC-12A
Location:
Newport Pass
Northing: 27.630893
Easting: 97.195882
Elevation: 5.60 ft m.s.l.
Depth Driven: 25 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
Shifting Dunes
PC-12A
Dunes
Dunes
Newport Pass


Benchmark
PC-12B
Location:
Corpus Christi Pass
Northing: 27.647940
Easting: 97.185093
Elevation: 5.75 ft m.s.l.
Depth Driven: 25 ft
G U L F
O F
M E X I C O
Shoreline
Beach
Timber Piles
Vegetation Line
PC-12B
Shifting Dunes
Dunes
207 ft
Dunes
Corpus Christi Pass


Benchmark
PC-13
Location:
Mustang Island State Park
Northing: 27.672367
Easting: 97.170277
Elevation: 5.54 ft m.s.l.
Depth Driven: 25 ft
G U L F
O F
M E X I C O
Shoreline
Timber Piles
Beach
2000 ft to South Fish Pass Jetty
Vegetation Line
Covered Tables
Dunes
38 ft
94 ft
6 ft
PC-13
Dunes
Mustang Island State Park Access


Benchmark
PC-14
Location:
Mustang Island State Park
Northing: 27.676050
Easting: 97.167612
Elevation: 5.55 ft m.s.l.
Depth Driven: 25 ft
Jetties
G U L F
O F
M E X I C O
Shoreline
Beach
Vegetation Line
150 ft
117 ft
Dunes
Light Pole
Trail
Mustang Island Fish Pass
Trail


Benchmark
PC-15
Location:
North of Fish Pass
Northing: 27.678377
Easting: 97.165850
Elevation: 4.75 ft m.s.l.
Depth Driven: 25 ft
Jetties
GULF
OF
MEXICO
Shoreline
Beach
150 ft
Vegetation Line
PC-15
2 mi to
Beach Access
Road 2
Dunes
Trail
Mustang Island Fish Pass
Trail


Benchmark
PC-16
Location:
North of Fish Pass
Northing: 27.683297
Easting: 97.163257
Elevation: 6.31 ft m.s.l.
Depth Driven: 25 ft
GULF
OF
MEXICO
Shoreline
Beach
2000 ft to
North Fish
Pass Jetty
1.65 mi to
Beach Access
Road 2
Vegetation
Line
PC-16
Dunes

Appendix E : Ground Photographs, April, 1996

This appendix contains ground photographs documenting the existing condition of the beach during the beach profile survey conducted in April 8, 9 and 10, 1996. The beach and surf conditions seen in the photographs are typical of the study area in the Spring or Summer.

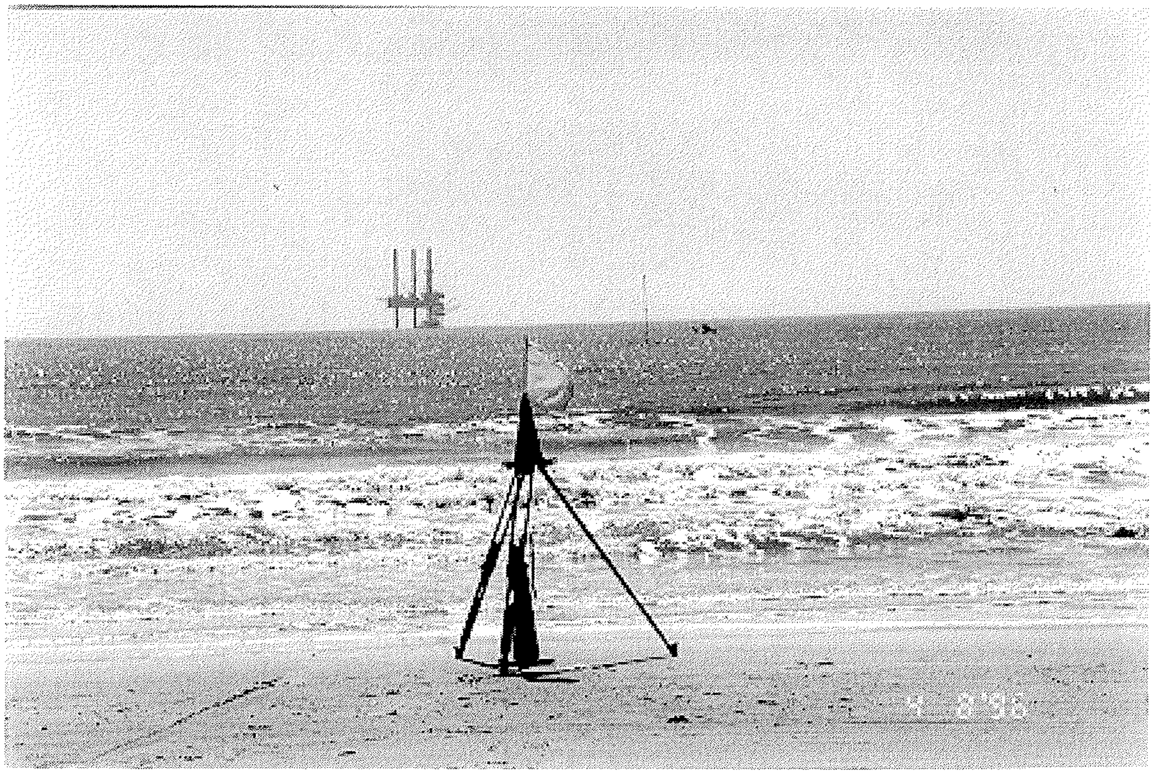


Figure E1. Sled in tow offshore of north Padre Island.

Figure E2. Total survey station. Note flag in dunes used for alignment by boat captain.

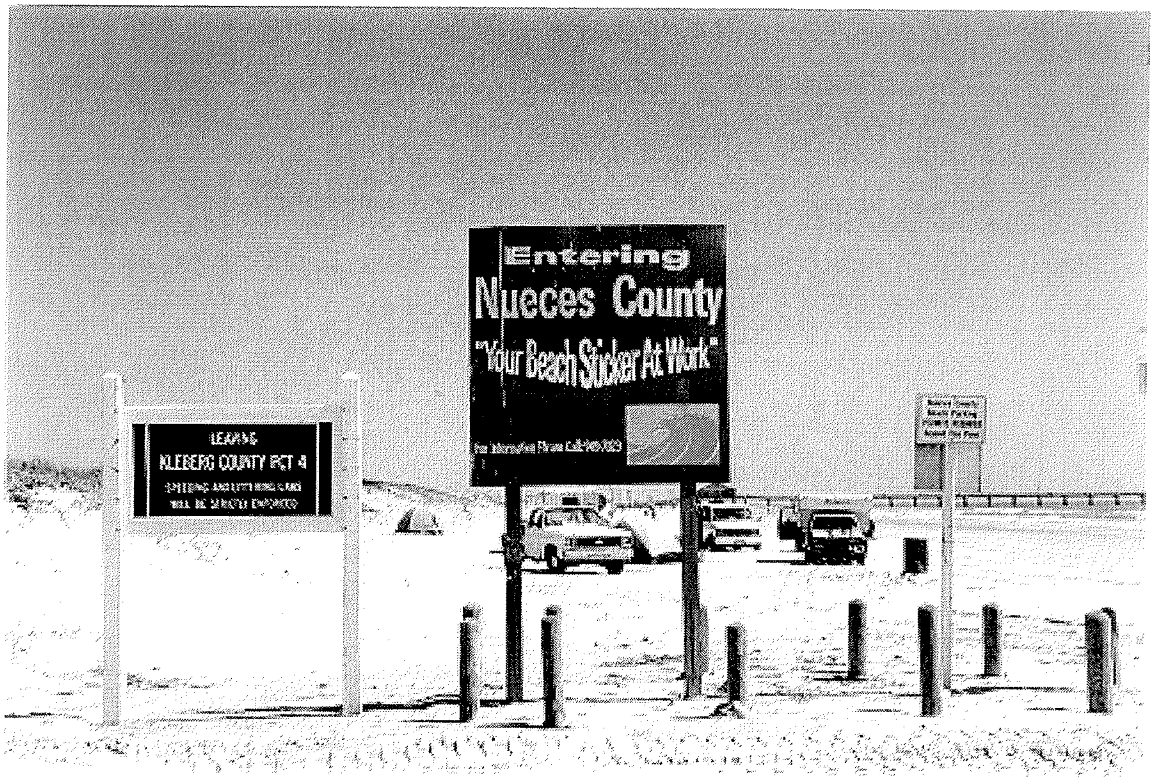


Figure E3. Near southernmost survey line (PC01). Note Bob Hall Pier in background.

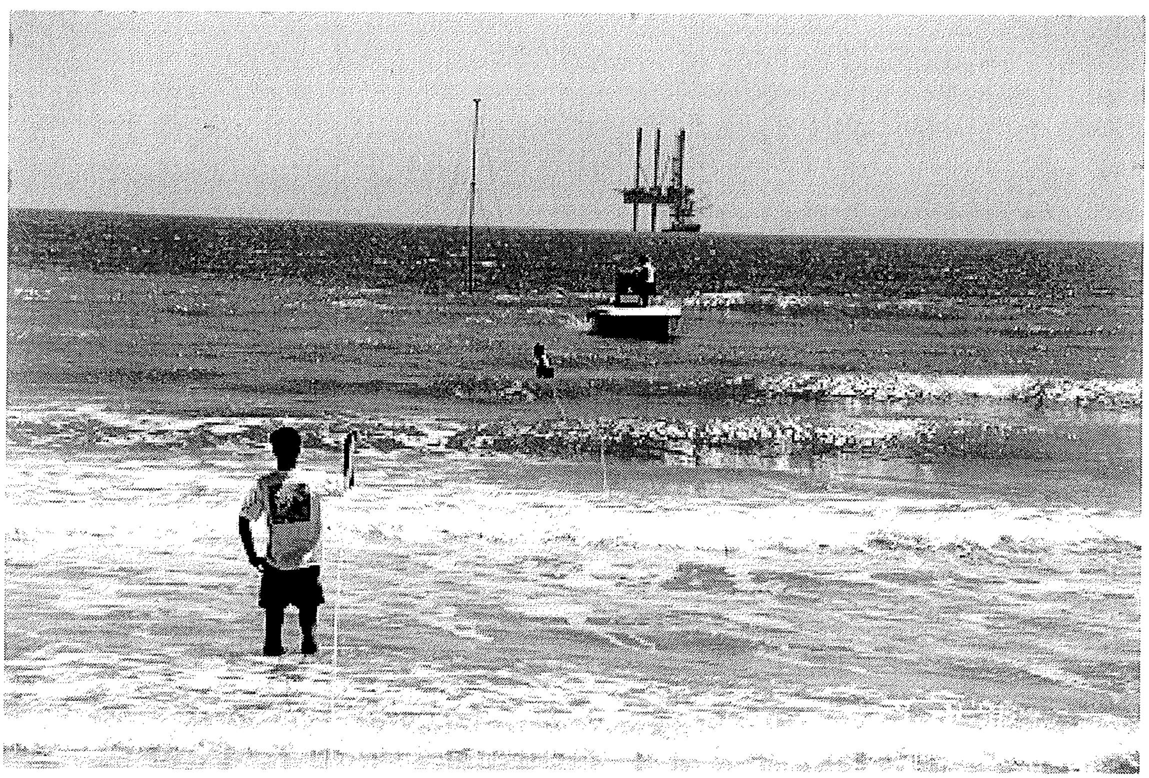


Figure E4. Towing sled onshore and passing the tow line to land.

Figure E5. Land portion of beach profile survey.

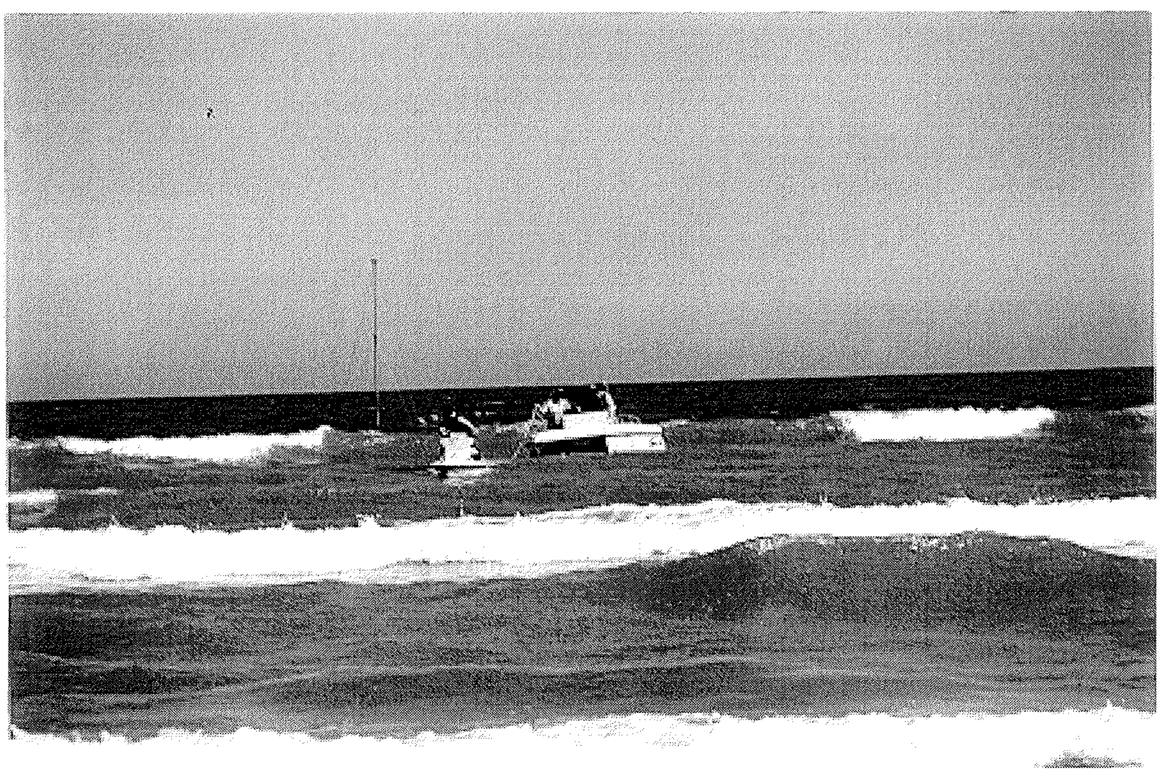


Figure E6. Transfer of tow line from boat to shore using a personal watercraft.

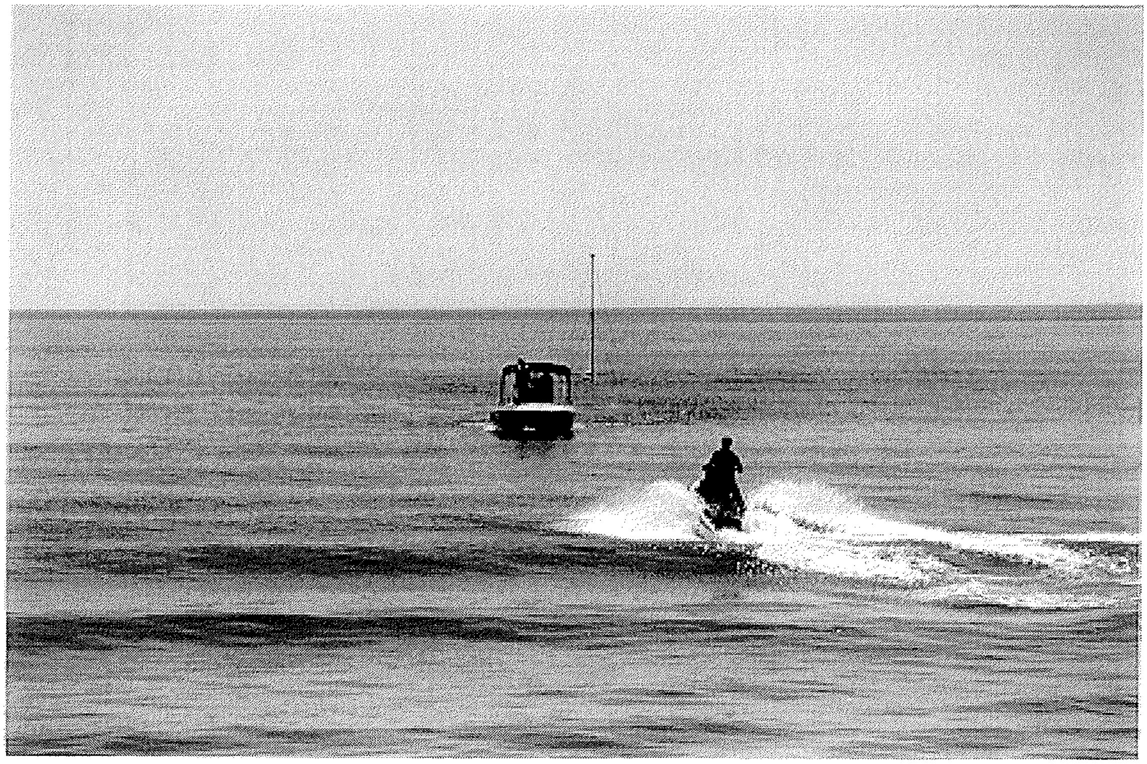


Figure E7. Particularly calm water was encountered during the morning of April 9.

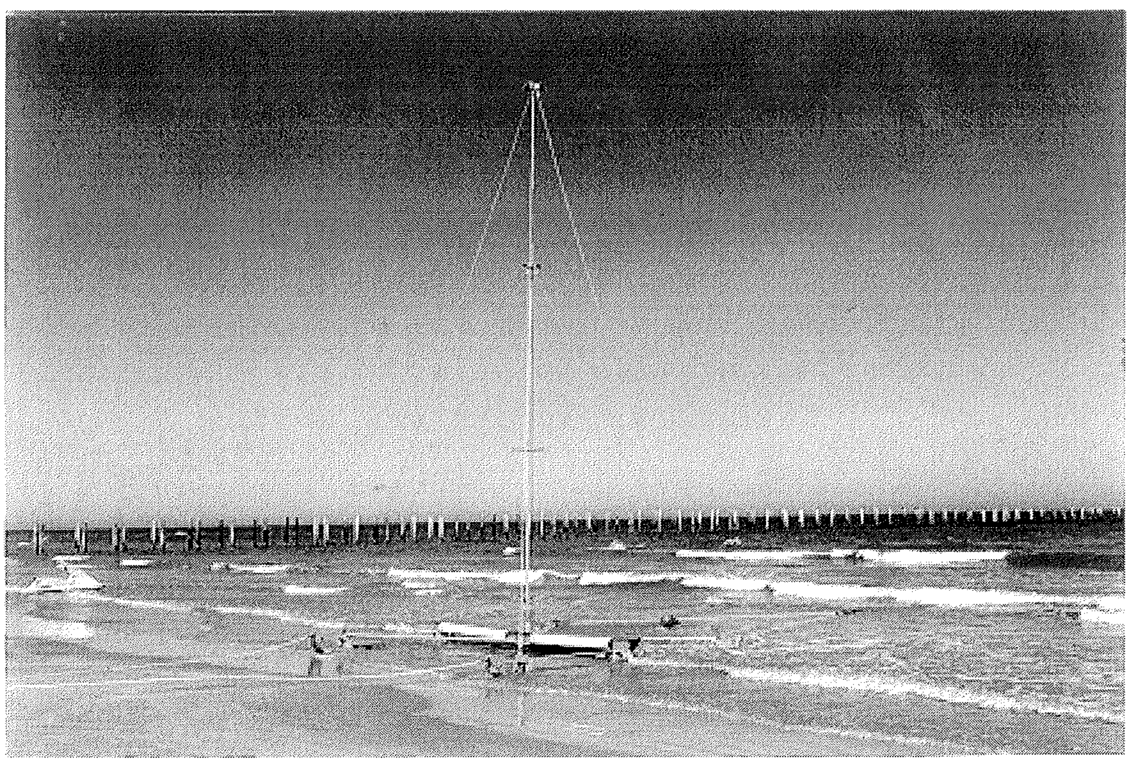


Figure E8. Sled being pulled onto the beach near Packery Channel at J. P. Luby Park. Note the surfing pier in the background.

Appendix F : Wind-Blown Sand Field Study

This appendix describes the background and field work of the wind-blown sand study conducted during April and May, 1996, at the Packery Channel project area for the shoreline processes and sand management component of the coastal processes assessment associated with re-opening of Packery Channel, Corpus Christi. This study was led by Dr. Ping Wang, who was conducting post-doctoral research at the Blucher Institute and was on temporary leave from the Geology Department, University of South Florida.

Although the problem has been identified during previous studies of stabilized Texas inlets (Kieslich 1977), the intrusion of wind-blown sand at inlets has been little studied, and limited recommendations and guidelines are presently available (USACE 1984). Horikawa *et al.* (1986) and Horikawa (1991) reviewed the literature on sand transport on a dry sand surface and found that transport rate is proportional to the cube of the wind speed or of the shear stress. Knowledge of wind-blown sand transport has been mostly applied to dune construction in coastal areas (e.g., Gage 1970, Otteni *et al.* 1972, Dahl *et al.* 1974, USACE 1984, and Texas General Land Office 1991). Little or no experience is available in applying wind transport knowledge to inlet design and maintenance. Lack of guidelines can only be resolved by conducting a field study at the site of interest. Therefore, intensive field measurements of wind-blown sand transport were conducted during the windy months of April and May, 1996, at the Packery Channel project area. The purpose of the field data collection was to obtain data to develop a practical site-specific empirical formula for predicting the rate of wind-blown sand transport.

The wind speed profile was measured with five micro-cup anemometers placed at elevations from 0.3 to 16 ft above the sand surface (Figure F1). Examples of the measured wind-speed profiles are shown in Figure F2. The detailed wind-speed profile allows reliable estimate of the shear velocity, which is traditionally considered to be a central parameter for predicting the wind transport. Note the accuracy of the predicted velocity profiles in Figures F3 and F4.

The rate of wind transport was measured using a horizontal sediment trap (Figures F5 and F9). The measurements were conducted for average wind speeds ranging from 7 mph to as great as 22 mph. Twenty-four measurements were conducted on a dry surface and one measurement on a wet sand surface (following a frontal passage). Seven of the 24 dry-surface measurements were conducted at the nearby (1 mile NE) Newport Pass. The measured rate of sand transport from 12 cu yd/ft/yr to as much as 150 cu yd/ft/yr. Qualitative field observations indicated that as little as 7% water content or modest vegetation coverage can significantly reduce the sand transport by wind.

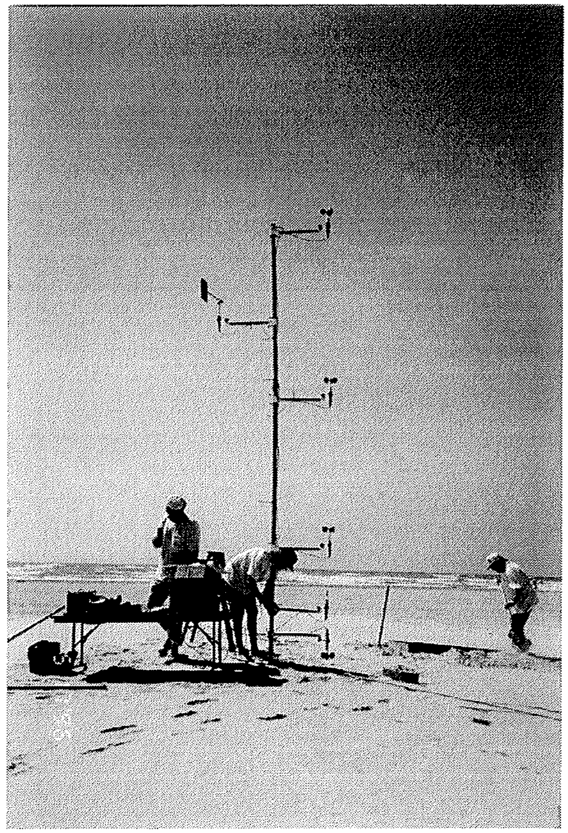


Figure F1. Anemometers set up near Packery Channel.

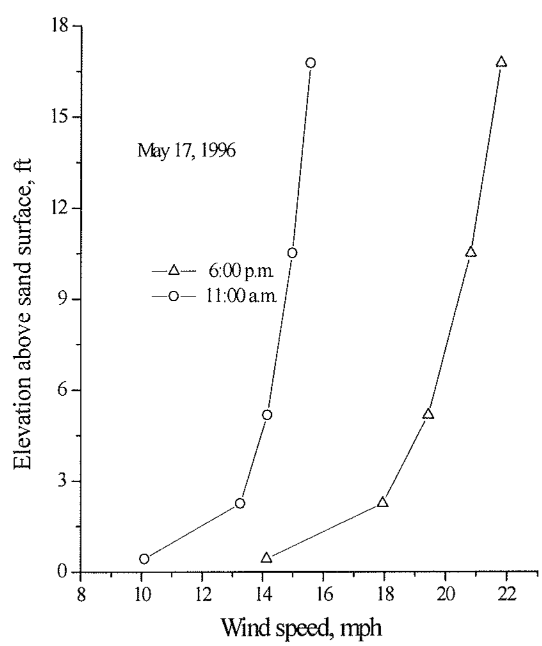


Figure F2. Examples of measured wind-speed profiles. Note the significant wind speed increase in the afternoon.

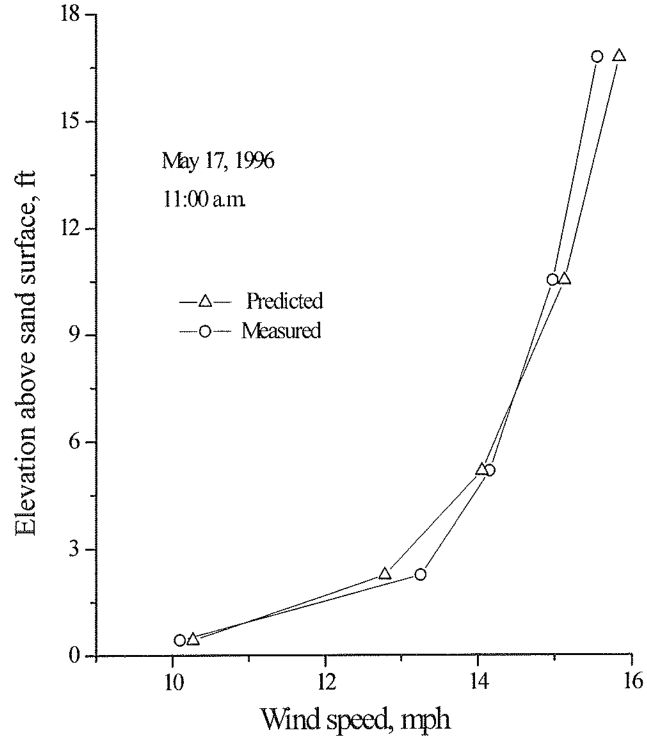


Figure F3. Measured and predicted logarithmic velocity profiles, May 17, 1996, morning.

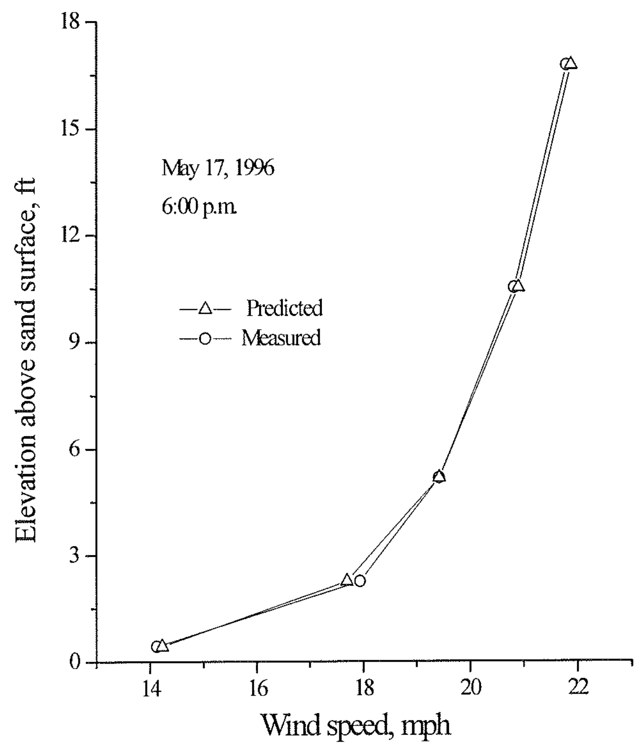


Figure F4. Measured and predicted logarithmic velocity profiles, May 17, 1996, early evening.

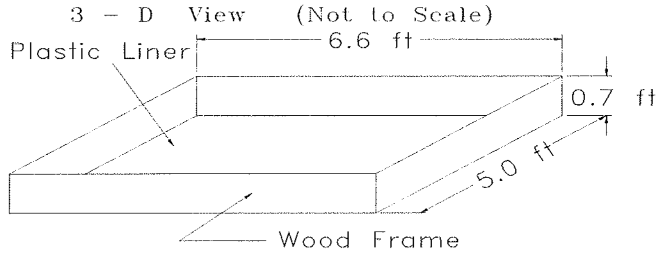


Figure F5. Design of the horizontal trap used to measure the rate of wind-blown sand transport.

The field measuring equipment was designed to be easily portable and capable of accurately measuring the transport rate during a 30- to 60-min period. The field measurement procedures were as follows:

1. Install the 5-anemometer array for wind-speed profiling.
2. Install the horizontal trap with water basin for total sand trapping.
3. Simultaneously start sand trapping and wind speed recording using a digital data-acquisition system developed at the Blucher Institute.
4. Simultaneously stop the sand trapping and wind speed measurement.
5. Remove the sand from the trap, and start again at Step 2.

Some of the field measurements were monitored by video camera. Different measuring durations, ranging from 20 to 85 min, were tested. Results indicated that the accuracy of the measurements was independent of measuring duration. Typically, 26 lb to as much as 308 lb of dry sand were trapped in the 6.6 x 4.9 ft area in an hour. The wind data and sand samples were taken back to the Blucher Institute laboratory for further analysis. The laboratory and data processing procedures were as follows:

1. Dry and weigh the trapped sand.
2. Analyze the sediment grain size of selected samples.
3. Convert dry sand weight into a transport rate.
4. Obtain average wind-speed profiles and least-square fit with the theoretical logarithmic curve and solve for shear velocity.
5. Compare the measured transport rate with the wind speed and develop an empirical formula to predict the sand transport rate by wind.

Previous studies, based in great part on laboratory experiments, indicated that the rate of sand transport is proportional to the shear velocity or to the wind speed (at a certain elevation) to the third power (Horikawa *et al.* 1986). The shear velocity has been incorporated into many empirical predictions, including those developed by Bagnold (1936, 1954), Kawamura (1951), Hsu (1971, 1977), and Horikawa *et al.* (1984). The detailed wind profiles measured in the present study allowed reliable estimate of shear velocity through least-square fitting of the theoretical logarithmic profile (Figure F2). The relationship between measured transport rate and best-fit shear velocity is shown in Figure F6. The relationship was found to be unsatisfactory for reliable transport rate prediction for the engineering application at hand. In addition, for practical applications typically only the wind speed is available, not the shear velocity.

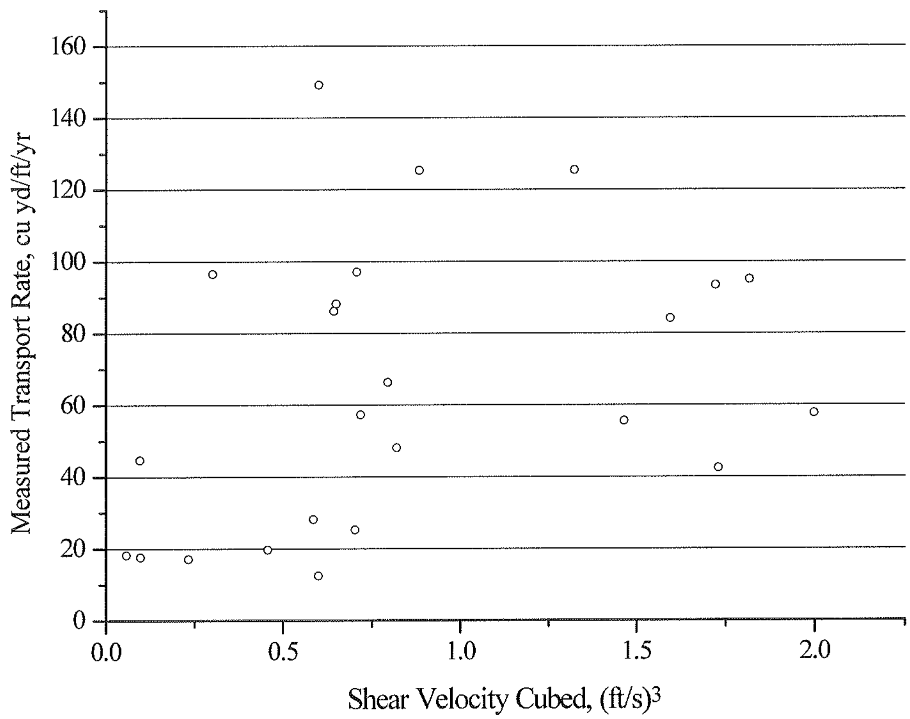


Figure F6. Measured transport rates versus the shear velocity cubed.

Field observations indicated that as the wind speed increased during the afternoon, small ripples developed on the sand surface, and at peak wind conditions (speeds around 22 mph) the ripples were replaced by plane bed conditions. The bed roughness length, which is critical for logarithmic curve fitting, is strongly controlled by surface bed form, which in turn determines the shear velocity. The poor correlation between measured sand transport rates and shear velocity is believed to be caused by a lack of representation of bed form in shear velocity formulations previously developed.

As an alternative to the shear velocity approach, a relationship between measured transport rate with wind speed at different elevations was examined (Figure F7). The correlation between the measured sand transport rate and wind speed at any elevation was significantly better than the correlation between the measured sand transport rate and shear velocity. The most consistent correlation was found using the wind speed measured 3.3 ft (1 m) above the sand surface.

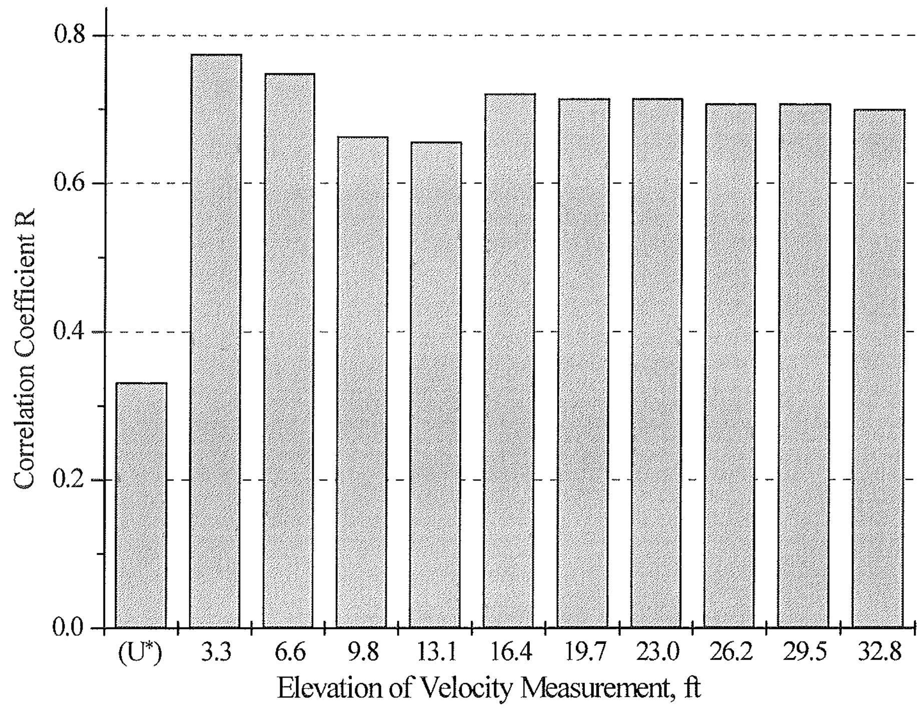


Figure F7. Correlation coefficients, R, between transport rate and wind speed for shear velocity, U^*, and at different elevations. The numbers on the horizontal axis represent the elevations at which wind speeds were obtained.

The first known transport rate formula, which was developed from measurements made at the mouth of the Columbia River, Oregon, used such an approach (O'Brien and Rindlaub 1936). In the present study a greatly improved correlation was found between the measured transport rate and wind speed at 3.3 ft (Figure F8).

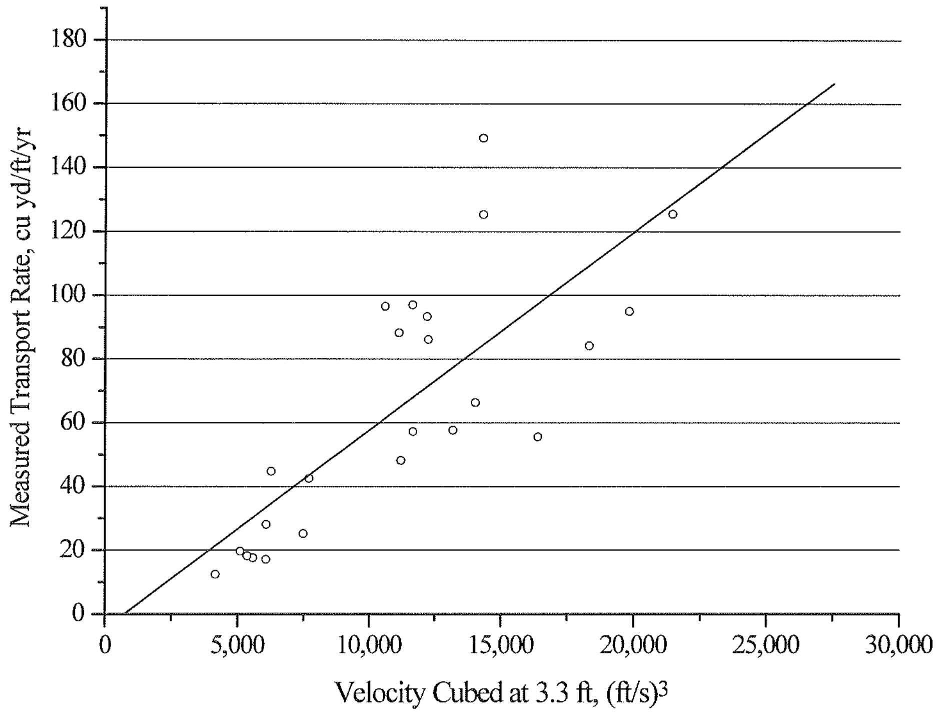


Figure F8. Relationship between measured transport rate and the wind speed cubed at 3.3 ft above the beach surface.

Remediation measures for wind-blown sand intrusion include installation of sand fencing, watering of the dry surface, planting vegetation (such as grass, shrubbery, and trees), and combinations. Sand fencing and watering of the dry surface were examined for rapid and short-term control of wind-blown sand transport. Vegetation is believed to be a long-term solution for much of the beach. Architectural design (blockage by structures) and sand handling and recycling will also play a role in the long-term strategy of reducing wind-blown sand intrusion and implementing a comprehensive sand management plan at the site.

Sand fencing has been widely applied to promote dune development. Previous studies of sand fencing conducted and applied along south Texas Coast include the work by Gage (1970), who experimented with fencing at Packery Channel, among other locations. Functions of sand fences in controlling wind-blown sand were discussed by Hotta *et al.* (1987, 1991) and Janin (1987). In the present study, a series of fence test sections were constructed with 25, 50, 70, and 90 percent spacing between the elements. The measurements were performed under strong winds (speeds as great as 22 mph on a dry sand surface. The 50% fence spacing was the most efficient for dune growth (Figure F10), which is in agreement with most previous studies. Severe scour at the toe of the fence and dune development behind the fence was also observed (Figure F11). Under strong wind conditions, equilibrium with a dune in front of and behind the fence was reached within days. Scour at the fence was severe, indicating a requirement for adequately driven fence posts.

The present and past studies indicate that under strong wind conditions, sand deposition behind fencing was promoted in the form of dune development. Even when vegetative planting is used to stabilize such dunes, single-row sand fencing alone is probably not effective for channel protection. This concept is supported by Gage (1970), who used the single-row fence-with-vegetation method at Packery channel to accumulate a maximum of only 8.1 cu yd/ft/yr of sand. Hotta *et al.* (1991) provided a comprehensive study of multi-row sand fencing, which indicated that two-row fences should be erected in regions with strong wind conditions.

Our field observations as well as those of previous studies have shown that slight water content in the sand will significantly reduce the magnitude of wind-blown sand transport. This phenomenon was noted by Otteni *et al.* (1972), who pointed out that the strong northerly winds associated with frontal passages in the area are usually accompanied by heavy precipitation and negligible sand transport. On the other hand, during his field study conducted along the Texas coast, Gage (1970) observed sand movement during periods of light rain and average wind velocities. Based on the present study, it is believed that a 10% water content in the sand can almost completely stop wind-blown sand even under very strong wind. This finding was confirmed with field measurements and qualitative observations in the Packery Channel area. Therefore, watering or mulching of the dry sand surface is recommended as a rapid and short-term method for wind-blown sand control. Another advantage of a moist sand surface is that growth of vegetation (grass, shrubbery, and trees) will be promoted. Further study is needed to quantify the rate of sand drying under different temperatures and wind speeds.

Vegetation has been proven by both agricultural engineers and coastal engineers to be the best long-term control method for dune strength and migration (Dahl *et al.* 1983) and has been shown to be an economically competitive and environmentally-sound control for wind-blown sand transport. The use of vegetation is a preliminary recommendation within a long-term plan for protection of the proposed Packery Channel and reduction of maintenance dredging. Based on comprehensive studies conducted on Padre Island, Otteni *et al.* (1972) and Dahl *et al.* (1974) concluded that bitter panicum and sea oats were the best adapted species of grasses for growth promotion and stabilization of local dunes. Although the use of exotic species was strongly discouraged, it was noted that other woody and herbaceous vegetation extensively existed on the Texas barrier islands prior to human exploration and colonization. Such vegetation may present aesthetically useful alternatives for wind-blown sand control used in combination with common indigenous grasses. Dahl *et al.* (1974) determined that indigenous grass plantings alone may accumulate an average of 5.2 cu yd/ft/yr of sand, and they reported sea oats plantings which resulted in the establishment of large dunes within five years.

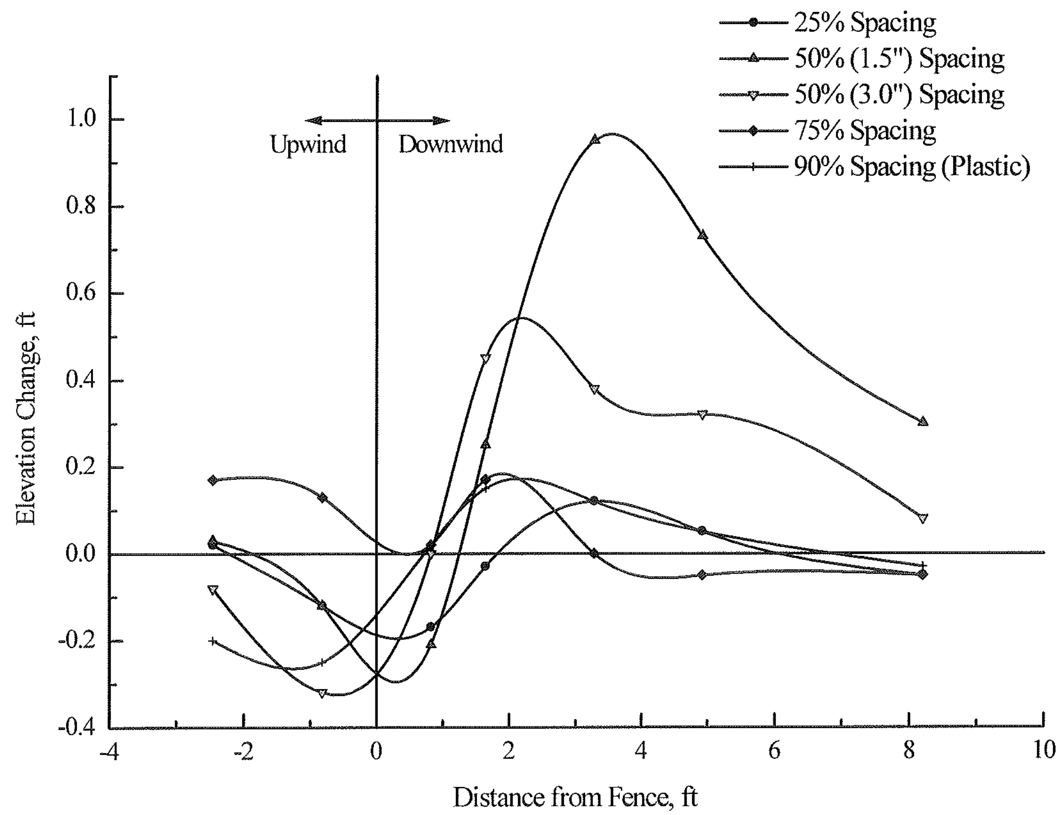


Figure F10. Sand dune development upwind of the sand fences 20 hours after installation.

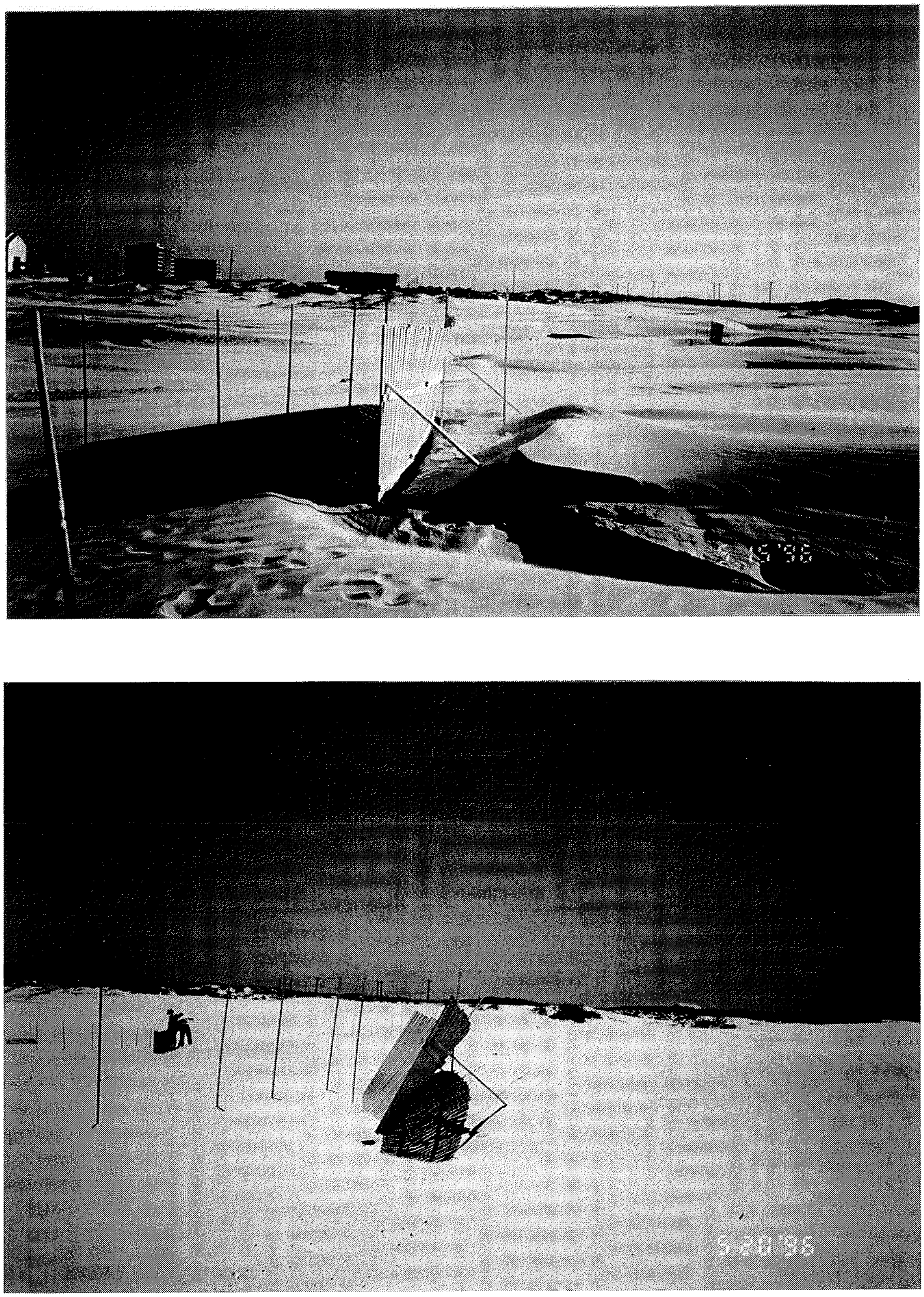


Figure F11. Sand fencing at Packery Channel. Upper photo: sand dune development adjacent to a sand fence a few days after installation. Lower photo: severe scour at the base of a fence.

Appendix G : Historical Tropical Storms

This appendix contains a summary of the documented historical storms which have impacted the southwest Texas coast. Sources of information for this compilation were NOAA, USACE, and Ellis (1988), as noted in Table G1.

Table G1. Summary of tropical storms which have impacted the Southeast Texas Coast.

Sources of Data:
- **a.** USACE. 1979. "Texas Coast Hurricane Study", USACE, Galveston District
- **b.** Ellis, M.J. 1988. "The Hurrican Almanac - Texas Edition."
- **c.** NOAA. 1987. "Hurricane Climatology for the Atlantic and Gulf Coasts of the U.S."
- **d.** Internet web site: http://wxp.atms.purdue.edu/hur_atlantic

SOURCE	NAME	DATE	LANDFALL LOCATION	INFORMATION
b		Fall, 1791		Flooded Padre Island killing 50,000 cattle on Ranch 15 miles SW of Nueces River
b		1828		Minimal hurricane caused high water at Corpus Christi.
b		8/4/1844		Corpus Chrisit has evidence of high winds, high tides, and turbulent bay.
b		8/1868		Port Aransas and Harbor Island flooded to depths of 2-3 feet. CC waterfront house destroyed.
b		7/4/1874		Damage to waterfront at CC where the shoreline was eroded back. CC experienced showers and very low tide.
b		9/5/1874		Destruction in CC by "most severe storm ever felt.....".
b		8/12/1880		In CC wind blew a "furious gale", barameter 29.52, 8-ft tide flooded area below bluff.
b, d	#5	8/12-21/1886		In CC: the Category 2 hurricane broke severe drought with 75 mph winds and 6" rain, NW winds caused water in bay to recede for 2 hrs.
b, d	#7	9/15-24/1886		50 mph winds in CC with light damage.
b, d	#2	8/22-29/1895		75 mph winds in CC.
a, b, d	#1	8/27/1900	Galveston	Category 4 hurricane that caused massive damage.
d	#2	7/2-10/1901	near Port Lavaca	Tropical Storm
a, b, d	#2	6/19-28/1902	Port Lavaca	Winds were 36mph in CC with 1.4" of rain. Storm caused minimal damage.
d	#1	6/25-30/1909	N. of Port Mansfield	Tropical Storm
d	#3	7/13-22/1909	N. of Freeport	Category 4 Hurricane in Gulf and Category 2 at landfall.
a, b, d	#2	9/5-14/1910	30mi. N. of Port Mansfield	Winds: 120 mph. Pressure: 28.5 inches. Over 9 " of rain with minor damage.
a, b, d	#5	10/11-17/1912	30mi. N. of Port Mansfield	Landfall at central Padre Isl. w/ 100 mph winds. The CC hurricane flag was raised (first time in 9 yrs.) Winds gusted at 51 mph. Little damage occured.
a, b, d	#1	6/22-28/1913	16mi. S. of Baffin Bay	Winds at 100 mph.
a, b, d	#4	8/12-19/1916	Baffin Bay	130 mph winds with central pressure of 28" and 9.2-ft surge. 1.58" of rain. Winds and high water destroyed boats and "every pier in CC Bay".
a, b, d	#2	9/2-15/1919	Baffin Bay	Category 2 hurricane. Full impact felt at Corpus Christi; 11.5-ft surge at Port Aransas. Maximum winds at 110 mph with pressure of 28.65". North Beach was completely submerged.
a, b, d	#1	6/15-26/1921	Palacios	Traveled due North past CC. Winds in CC were 68 mph. Category 1 at landfall.
b, d	#1	9/6/1925	Brownsville	Tropical Storm. CC had 42 mph winds and 1" of rain.
a, b, d	#1	6/27-29/1929	Port Lavaca	In CC, winds were not strong, beneficial rain of 1.14". Category 1 at landfall.
b, d	#1	6/25-28/1931	Baffin Bay	Tropical Storm. In CC, no high winds, but rained as much as 8.03".
b, d	#2	8/12-15/1932	btwn Galveston & Freeport	High tides in Corpus Christi. Category 2 at landfall.
a, b, d	#4	8/28-9/5/1933	30mi. S. of Port Mansfield	Tropical Storm with max. winds of 40 mph at landfall. A 5-ft surge at North Beach. The causeway from Flour Bluff to Padre Isl. was destroyed.
a, b, d	#3	7/21-26/1934	Rockport	Winds were 56 mph in CC. Barometer fell to 29.12". A 10.2-ft surge recorded on St Joseph's Island. Category 1 at landfall.
a, b, d	#3	6/26-28/1936	Aransas Pass	Winds were 36 mph at CC from NW causing tides to blow out. Category 1 hurricane.
b, d	#14	9/10-14/1936	70mi. S. of Corpus Christi	Tropical Storm with maximum winds at 40 mph and 1.39" of rain.
b, d	#3	8/23-28/1938	Mexico	In CC, rain squalls and 31 mph winds. Category 1 hurricane with a path south of TX.
a, b, d	#2	9/16-25/1941	Freeport	90 mph winds and tide of 9.9 ft. Category 1 hurricane.
a, b, d	#2	8/21-31/1942	Port Lavaca	In CC, tides rose 3ft above MSL. 72 mph winds. The storm reopened the two 1933 channels of CC Pass. Category 1-2 at landfall.
d	#2	7/19-22/1945	Pt. Mansfield/Baffin Bay	Tropical Storm with maximum winds at 35 mph.
a, b, d	#5	8/24-29/1945	Matagorda Isl.	Category 4 hurricane skirted coast along Nueces Co. In CC, 70 mph winds and 4.14" of rain.
b, d	#1	7/31-8/2/1947	Brownsville	Tropical storm, highest winds in CC were 33 mph with 3.28" of rain.
a, b, d	#10	9/27-10/6/1949	Matagorda	In CC, most washover channels on Padre opened, and large amounts of rain stood behind dunes. CC max. winds were 23 mph. Category 4 at landfall.
b, d	HOW	10/1-4/1950	150mi. S. of Brownsville	Tropical Storm with 39 mph winds in CC. Tides rose 3.5'-4' on Padre Island, parts of hwy at Gulf Park were destroyed.
b, d	GLADYS	9/4-6/1955	Mexico	CC received 7.68" rain in 24 hrs. Tides rose 4.5' on N. Beach. Minimal damages. Category 1 hurricane at landfall.
b, d	AUDREY	6/25-29/1957	near Cameron	Category 2-4 hurricane traveled north from Mexico to TX/LA border. In CC, minor damage included Park Rd washed out between CC Pass and Packery
a,b, d	ELLA	8/30-9/6/1958	Corpus Christi	Hurricane that weakend to a small Tropical Storm in the Gulf prior to landfall.

Table G1 (con't). Summary of tropical storms which have impacted the Southeast Texas Coast

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SOURCE	NAME	DATE	LANDFALL LOCATION	INFORMATION
b, d	HOW	10/1-4/1950	150mi. S. of Brownsville	Tropical Storm with 39 mph winds in CC. Tides rose 3.5'-4' on Padre Island, parts of hwy at Gulf Park were destroyed.
b, d	GLADYS	9/4-6/1955	Mexico	CC received 7.68" rain in 24 hrs. Tides rose 4.5' on N. Beach. Minimal damages. Category 1 hurricane at landfall.
b, d	AUDREY	6/25-29/1957	near Cameron	Category 2-4 hurricane traveled north from Mexico to TX/LA border. In CC, minor damage included Park Rd washed out between CC Pass and Packery Channel.
a,b, d	ELLA	8/30-9/6/1958	Corpus Christi	Hurricane that weakend to a small Tropical Storm in the Gulf prior to landfall.
a,b, d	DEBRA	7/22-27/1959	Galveston	In CC, 2" rain and minor street flooding.
d	TS#1	6/22-29/1960	Corpus Christi	Tropical Storm with maximum winds at 40 mph.
a, b, d	CARLA	9/3-15/1961	Port Lavaca	Winds of hurricane velocity from Sabine River to Corpus Christi; Bob Hall Pier demolished. Surge levels ranged from 6.8'-8.8'. Dunes on Mustang Isl. eroded as much as 15 ft. Category 4-5 at landfall.
a, b, d	CINDY	9/16-19/1963	near Galveston	Tropical Storm looped south over land towards CC.
d	ABBY	8/5-8/1964	Matagorda	Tropical Storm.
a, b, d	BEULAH	9/8-21/1967	Brownsville	Winds of hurricane velocity as far north as Matagorda Bay; end of Bob Hall Pier destroyed. T-heads were under water. Category 5 at landfall.
b, d	CANDY	6/22-26/1968	Corpus Christi	Tropical Storm with 62 mph winds in CC. Bridges at packery channel & CC Pass washed out.
a, b, d	CELIA	7/30-8/5/1970	Corpus Christi	Estimated 180 mph winds at CCi with 9-ft surge. Category 3 at landfall.
a, b, d	FERN	9/7-13/1971	Matagorda	Tropical Storm made landfall at Matagorda and skirted coast towards Pt. Aransas. In CC, 70mph winds, 3-4' tides, waves crashed over T-heads. Up to 14' of rain.
d	EDITH	9/5-18/1971	---	Traveled from Mexico/TX border to Louisiana/TX border. Category 1-2 hurricane as passing by CC.
d	DELIA	9/1-7/1973	Freeport	Tropical Storm looped around Freeport.
b, c, d	ALLEN	8/10/1980	NE of Brownsville	92 mph winds at Corpus Christi; 8-9 ft surge. Sustained winds of 47 mph for 24 hrs. Bob Hall Pier demolished. Category 4 at landfall.
b, d	JEANNE	11/7-16/1980	---	High swells on Padre Island; severe beach erosion in areas already battered by Allen
d	ALLISON	6/24-7/1/1989	Galveston	Tropical Storm developed off CC coast and traveled north to Galveston.
d	ARLENE	6/18-21/1993	Baffin Bay	Tropical Storm caused over 1-ft of rain in South Texas.