US Army Corps
of Engineers

Galveston District
Southwestern Division

Modeling Study & Results

PACKERY CHANNEL – PROJECT DESIGN

CORPUS CHRISTI, TEXAS

Prepared by:

Chris Reed, Ph.D
Alan Niedoroda, Ph.D.
Mark E. Mazoch, P.E.

URS/Dames & Moore

9801 Westheimer, Suite 500
Houston, Texas 77042

FEBRUARY 24, 2003

TABLE OF CONTENTS

Section **Page**

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES

1.0 A SUMMARY OF FINDINGS

Several factors affecting the feasibility of a re-opened Packery Channel have been re-evaluated in support of a Design Documentation Report (DDR). This study followed recommendations in the Project Report (Dames & Moore, 2000) for enhancing the modeling analyses. These recommendations consisted of using improved wave hindcast data as input to the sediment transport (littoral drift) and shoreline modeling, calculating severe wave heights in the channel (especially under ebb flow conditions), and revising the inlet stability analysis. This study has been carried out with the following findings:

- The estimated channel dredging is 38,000 cy/yr and the variability is shown as a cumulative frequency chart (Figure 5.10). This estimate assumes that wind blown sand does not contribute to the dredging requirements and does not include potential channel shoaling due to cyclonic storms. It is recommended that the channel bathymetry be monitored on a regular basis and after major storm events so that dredging is scheduled before shoaling becomes a navigation hazard.

- The sand management options for Packery Channel are difficult to develop because the direction of sand transport is not well defined. The proposed Packery Channel lies near a nodal point in the sand transport system, and there is conflicting evidence as to the long-term net transport direction. The annual average required mechanical bypassing of sand to maintain the current shoreline position is 160,000 cy/yr. The variability about the annual average is expressed in a cumulative frequency chart (Figure 5.8). We expect that there will be extended periods, lasting one or more years, where there will be alternately up- and down-coast net sand transport.

- During periods when the net transport direction proves to be to the north, the seawall will be naturally protected by the generation of a fillet along the south jetty. It may be possible to let the fillet build out to a prescribed location before sand management is invoked, thus providing additional protection for the seawall. However, this approach will be at the expense of some erosion on the north beach. During periods when the net transport direction is to the south, more aggressive sand management will be required to keep the downdrift erosion from encroaching on the seawall.

- It is recommended that a monitoring program, consisting of regular beach profiling in the vicinity of the jetties, be maintained to provide data on the net transport direction and the need for sand bypassing. Also, monitoring after major storms should be conducted.

- Monitoring should be done seasonally, and after major storms initially. The monitoring interval can be lengthened but should be at the least, annual. It is also recommended that any pre-construction sand bypassing configurations be designed to accommodate bi-directional sand bypassing.

- Because the direction of the net littoral drift transport cannot be reliably predicted, the placement of the beach fill material, obtained from the initial construction dredging, is not obvious. Model analysis indicates that, for reasonable fill lengths along the beach, the effects of the beach fill will last about 3 years after which the shoreline tends to return to its original position. There is interest in placing the material in front of the seawall to provide additional protection from major storm events. If the long-term net transport is to the north, then this would only have been useful to provide initial protection, since the development of an updrift fillet would provide similar protection. However, if the transport is found to be from the south, the fill will provide protection that would otherwise not be available. Two options for placement are recommended. The first places the sand on the south side, in front of the seawall. The second option places the sand, equally to the north and south of the jetty.

- Under normal conditions, sand bypassing can be conducted on either continuous, yearly or biennial schedules without significant impacts to the downdrift beach. However, channel dredging will likely be required on an annual basis, if not more frequently, because the estimated sedimentation rate in the channel could fill the channel within one year. A small dedicated dredge could be an economic approach for regular dredging.

- Adequate maintenance dredging and sand bypassing combined with a rigorous beach and channel monitoring program can minimize beach erosion in the vicinity of the channel.

- Analyses of common intervals from three hindcasts and two measured wave data sets show that directional agreement is moderate to good while wave height agreement is on the order of 15 % to 25 %. The results of these comparisons appear to be a measure of an underlying uncertainty concerning the wave heights in the project area;

- Comparisons of ‘updated’ Wave Information Study (WIS) wave hindcast wave heights with two measured wave data sets show the measured heights straddling the hindcast wave heights. This indicates that these WIS data are as good a measure of the wave climate as are available for the Packery Channel area;

- There is conflicting evidence, in terms of geological data and wave/wind data, that makes prediction of the net longshore sediment transport uncertain. This uncertainty has been incorporated into the transport and shoreline migration analysis;

- The GENESIS model has been successfully calibrated to historical conditions at nearby Mansfield Pass and used to evaluate effects of the proposed jetties at Packery Channel on shoreline migration and on sand management requirements;

- The littoral transport modeling study is based on the GENESIS model and has been successfully calibrated to historical conditions at nearby Mansfield Pass. The calibrated model has been used to estimate the effects of the proposed Packery Channel jetties will have on shoreline migration, required sand management and channel dredging.

- The effects of significant tropical storms, hurricanes or other extreme events are not included in the modeling analysis and predictions. These events could significantly impact channel siltation resulting in emergency dredging projects and beach erosion.

- The downdrift beach is estimated to recede up to 350 ft near the jetties if no sand management is applied. The extent of the erosion extends 10,000 ft down the shoreline.

- It is recommended that spur jetties not be included in the jetty design due to the sand management problems that are likely to occur.

- The options to extend the jetties an additional 150 ft past the 1400 length is not recommended because it was found that the additional length is not expected to reduce maintenance dredging.

- The annual maintenance dredging volumes reported herein do not include contributions to wind blown sand. It is assume that the control of wind-blown sand is included in the project design.

- The proposed channel configuration is expected to have marginally better stability characteristics than those at Mansfield Pass and the channel will likely trap sediment available at the jetty tips.

2.0 INTRODUCTION

Packery Channel was a tidal pass opening to Corpus Christi Bay that closed sometime after a 12-foot deep boat channel was dredged from Aransas Pass to the bay in 1912. Since that time, it has opened only temporarily after intense storms. A recent study (Kraus and Heilman, 1997) has examined the feasibility of re-opening and maintaining Packery Channel. Although this study resulted in a conceptual design and a forecast of maintenance dredging requirements, issues have been raised about the wave data used in the numerical modeling analyses to forecast shoreline changes and future channel sedimentation (Dames & Moore, 2000). This report results from a restudy of the proposed Packery Channel with more recently available data.

The location of Packery Channel on the central Texas coast is shown on Figure 2-1. This figure depicts several features and locations that are referenced in this report. These include the locations of three updated 1976-1995 WIS hindcast stations, and National Oceanic and Atmospheric Administration (NOAA)/National Cliamatic Data Center (NCDC) Buoy 42020, and LaTex Mooring #1. The 'updated' WIS stations are the locations where wave hindcasts have been conducted by the United States Army Corps of Engineers (USACE). These hindcasts were reviewed and updated in the year 2000, and the title 'updated' is appended to distinguish these data from earlier data that were considered less accurate. Figure 2-2 shows the proposed arrangement of Packery Channel and its protecting breakwaters.

Several studies have been performed and reports written about the feasibility of opening and maintaining Packery Channel. The two-part series of reports prepared for Naismith Engineering, Inc. by the Conrad Blucher Institute for Surveying and Science in 1997 is thorough. A detailed description of the extent of the proposed project, the supporting and previous studies, existing conditions, and their feasibility analyses are included in the report by Kraus and Heilman (1997).

3.0 WAVE DATA ANALYSIS

The analyses for sediment transport, shoreline migration, inlet stability and wave steepening discussed in the subsequent sections of this report require a representative wave data set. A number of wave data sets, both hindcasts and measurements, have been identified in the Packery Channel area, and are listed in Table 3.1. The WIS data sets covering the period 1976 – 1995 are the most recent available. These data are wave hindcasts produced by the USACE and have been recently updated. As a first step, we reviewed these data and compared them with others to decide whether the 'updated' WIS data are adequate for the subsequent analyses made in our study.

We have made a number of comparisons between the measured and predicted data sets to determine the suitability of the data for the modeling analysis.

Table 3.1 Wave Data Sets Used in the Wave Analysis

Data Set	Water Depth (ft)	Location	Period of Record	Source
OCTI Hindcast	32	3000 ft offshore Packery Channel	1970 - 1979	Shiner Moseley & Associates, Inc.
WIS 1087	59	27.75N 97.00W	1976 - 1995	WIS web page
WIS 1088	90	27.50N 97.00W	1976 - 1995	WIS web page
LaTex Mooring #1	70	27 15.39N 97 14.81W	1992 – 1994	Texas A&M University
WIS 1089	70	27.25N 97.25W	1976 – 1995	WIS web page
Buoy 42020	255	27.01N 96.51W	1990 - 2000	NCDC
WIS 1099	465	27.00N 96.50W	1976 – 1995	WIS web page

Two locations provided a comparison between a 'updated' WIS wave hindcast time series and a corresponding time series of wave measurements. An additional comparison was made between the 'updated' WIS data and the Offshore and Coastal Technologies Incorporated (OCTI) hindcast data, which were used in the previous modeling analysis.

Data from 'updated' WIS 1099 NOAA/(NCDC) Buoy 42020 provide a comparison of hindcast and measured wave data in relatively deep water. The data from corresponding periods (1990 -1995) were used to make the comparisons. Figures 3-1 and 3-2 show the wave roses for each of the two sites. The wave roses show that the general directions are in good agreement. The ‘updated’ WIS data wave directions are more centered on the southeast direction, while the buoy data have larger waves coming from a slightly more southerly direction and smaller waves coming more from the north. Figure 3-3 shows a comparison of the wave heights for each station. The points on this figure represent daily comparisons between the data sets. The longer straight line represents where all the points would plot if there were a perfect comparison. The shorter trend line shows that, over the 10-year period of overlap, the measured waves are, on the average, 25% higher than those of the ‘updated’ WIS hindcasts.

During 1992 – 1994, the U.S. Minerals Management Service (MMS) supported a study of the oceanography of the continental shelf of Louisiana and Texas. This project, named LaTex, included a number of instrument moorings, some of which accomplished directional wave spectra measurements using pressure and wave orbital velocity measurements. One of the ‘updated’ WIS hindcast data stations (Number 1089) is located near the LaTex Mooring #1 position and provides a good comparison of data in shallower water. Only the data where both systems produced equivalent time series were used. Figures 3-4 and 3-5 show the wave roses for each of the data sets. A comparison of average wave conditions is shown in Table 3.2 below.

Table 3.2 Comparison of ‘Updated’ WIS and LaTex Wave Data

Period	Wave Height (m)		Wave Direction (deg)		Wave Period (s)	
	LaTex	WIS 1089	LaTex	WIS 1089	LaTex	WIS 1089
1992-1994	0.72	0.85	116.56	115.67	6.32	5.80

The wave roses show that the waves are generally in the same direction, from the southeast, with the measured data having a slightly broader distribution in directions. The average values shown in Table 3.2 indicate that the ‘updated’ WIS data over-predict the measured LaTex wave heights by about 13%, while the predicted period is shorter. However, the LaTex measurements may have been biased downward due to vertical attenuation acting on the bottom-mounted pressure meter. The mean wave directions are within one degree of each other.

Figure 2-1 shows that there are two ‘updated’ WIS stations in the vicinity of the proposed Packery Channel, # 1087 and # 1088. We have selected to use the data from Station # 1087 since it is in shallower water. In the previous modeling analysis of the Packery Channel littoral drift and shoreline migration (Kraus and Heilman, 1997), a hindcast from OCTI was used. The data set corresponds to a water depth of 32 ft, which was about 3000 ft offshore. In order to compare the “updated “ WIS hindcast data to the

OCTI data, the 'updated' WIS Station # 1087 data were transformed to a water depth of 32 ft using a shoreline azimuth of 25 degrees. The 'updated' WIS data and corresponding OCTI data are summarized in Table 3.3 below.

Table 3.3 Comparison of 'updated' WIS # 1087 and OCTI Hindcasts

Year	Height (ft)		Period (s)		Direction (deg)	
	59 ft	32 ft	59 ft	32 ft	59 ft	32 ft
76-95	3.1	2.62	6.00	5.74	122.01	120.96
76-79	3.1	2.60	6.03	5.76	119.53	118.73
80-95	3.1	2.62	5.99	5.74	122.63	121.51
OCTI Wave Hindcast Data (water depth of 32 ft)						
76-79		1.7		4.24		112.42

The effects of the transformation of the 'updated' WIS data from 59 ft to 32 ft included the following: a reduction in average wave height, by about 16%, a shortening of the period, and, a slight turning to the south (the direction from which the waves are coming swings to the north) of about a degree. A comparison during the overlapping time periods between the 'updated' WIS and OCTI data shows significant differences in the average wave statistics. The 'updated' WIS data wave height is about 1.54 times higher than the OCTI hindcast wave heights. The wave periods in the WIS data are also longer than those in the OCTI hindcast, and the average wave propagation direction is 6 degrees more to the north. Wave roses for both data sets are shown in Figures 3-6 and 3-7, confirming the general difference in wave direction. Assuming a shoreline azimuth of 25 degrees in the Packery Channel, the OCTI data set tends to produce waves that are more shore-normal than those in the 'updated' WIS data set. The 'updated' WIS data set predicts waves to come more from the south, which would tend to increase the northward amount of littoral drift.

In summary, the direction of the 'updated' WIS data generally agree with that of the measured data in the area of Packery Channel. Within identical time periods the corresponding wave heights are lower than the NOAA/NCDC buoy data but higher than the LaTex Mooring data. This straddle is probably a measure of the certainty that can be expected from any of the wave time series data. The 'updated' WIS # 1087 data set predicts larger, longer waves than the OCTI hindcast data set, with waves propagating in a more northward direction. As will be seen in subsequent sections, the use of the WIS data tends to yield a northward net transport over the 20-year period 1975 – 1995.

4.0 LARGE –SCALE WAVE CLIMATOLOGY

4.1 CONSIDERATIONS ON WAVE DIRECTION

In a previous analysis of sand transport at the proposed Packery Channel (Kraus and Heilman, 1996), some reliance was placed on geological evidence to support a conclusion that the dominant future net littoral transport would be down-coast (southwestward). This is relevant in analyzing the stability of a reopened channel and for planning sand management. We have re-examined the geological data thought to support the idea that there is a pattern of converging annual net littoral transport located southwest of Packery Channel in order to try to resolve the conflicting conclusions from geological and wind and wave climatology data. The geological evidence comes from studies of the occurrences of certain trace mineral grains in the sands, from shells that are common on the beach about 20 miles southwest of Packery channel, and from consideration of the wave, current, and other processes that produce deposits of beach sand.

The first trace mineral study was conducted by Fred Bullard (Bullard, 1942) who separated heavy minerals from 55 sand samples collected on Texas beaches and coastal rivers. Thirty-eight of the samples came from beaches between Brownsville and Port Arthur. The rest were collected from the Rio Grande, Nueces, San Jacinto, Colorado, and Brazos Rivers. The samples were analyzed to determine the relative abundance of 15 heavy minerals and mineral classes. “Three groupings of the heavy minerals were recognized for the purpose of comparison base primarily on the relative occurrences of the following: basaltic hornblende and pyroxene, green hornblende, and more durable minerals including garnet, tourmaline, rutile, zircon, and staurolite” (op cit.). The mineral suite rich in basaltic hornblende and pyroxene is thought to originate from the weathering of volcanic bedrock along the Rio Grande River. This suite can be followed from the mouth of this river northward along Padre Island a zone adjacent to Baffin Bay (~27.3^{o} N) beyond which it is diluted by grain populations thought to be representative of the Nueces and other central Texas coast rivers. Bullard (1942) considers the Colorado River to be a significant source of some of the heavy minerals on the central Texas coast.

Van Andel and Poole (1960) published a second study of the distribution of heavy minerals. Although similar to the Bullard study, the scope was larger with samples extending offshore across the continental shelf and eastward to the Mississippi Delta. Their results agree with the earlier study on the extent northward that the heavy mineral suite of the Rio Grand can be followed up Padre Island before it becomes diluted with sand from other sources. They identify another distinctive heavy mineral suite attributed to the Mississippi River and other suites associated with the rivers of the central Texas coast. The strongest indicators of the source rivers are in halo areas around the mouths of the Rio Grande and Mississippi Rivers. The Mississippi River halo extends across the continental shelf and the other halo is more restricted. Their conclusions about the central Texas coast are at variance with those of Bullard. Van Andel and Poole conclude that the sediment now on these beaches has a complex origin involving both local and distant rivers and transport paths that go so far back as to include coastal systems active thousands of years ago when sea level was much lower. They also point out that sediments on the continental shelf are exchanged with beach sediments in this general area so that modern and ancient distributions of the trace mineral grains contribute to their present distributions.

A heavy mineral suite from the Colorado River can be traced along the shore to the area of the Rockport bays but elsewhere the beach sands appear to be mixtures of recent and ancient sources, many of which are now located offshore on the continental shelf. There is general support for the idea of a convergence of littoral transport cells on the central Texas coast but they cannot distinguish the location more specifically than somewhere between Baffin Bay and Rockport.

Two other studies argue for the existence of a transition zone in beach sediments marking a point of littoral drift convergence a short distance down-coast from the Packery channel. Hayes (1997) found that the beach sand of central Padre Island is a mixture of coarser sand from the Rio Grande with finer sand from the other rivers (e.g. Nueces, Brazos, etc.) to the north. He identifies a transition zone and places it in an 8-mile beach stretch between approximately 27.05^{o} and 27.15^{o} N (~ 32 to 40 mi. SW of Packery Channel). This is close to where Bullard found that the heavy mineral suite of the Rio Grande became diluted with grains from other river sources.

Watson (1971) also postulates a relatively narrow transition zone between net up and down-coast littoral transport based on the common occurrence of shells and shell fragments on the beach. His location is between 15 and 25 miles southwest of the Packery channel. In the northern portion of this zone there are large populations of small bivalves (clams) living in the lower portion of the beach face. These do not appear to be carried further down-coast. Instead, other shells, generally of great age (> 1,000 yr.) occur indicating that they have been washed ashore from the adjacent continental shelf. In the stretch of beach extending 10 miles northward from a point 40 miles south of the Packery channel, these old shells become progressively more fragmented and worn in a northward direction. Watson further points out that the stretch of coast where high shell concentrations are found is adjacent to the Central Texas Aeolian Sand Plain. Strong onshore winds, combined with limited vegetation cause continued transport of beach sand many tens of miles inland. Watson (1971) endeavored to confirm the association of this zone of high beach shell deposits with the net littoral transport convergence postulated by Hayes (1965) by analyzing the wind record from Corpus

Christi between 1951and 1960. From this he concluded that a littoral drift convergence on the central Texas coast would occur near Aransas Pass. He points out that Price (1933) came to a similar result by analyzing Corpus Christ winds in the period 1923 – 1930.

Carothers and Innis (1962) also based an analysis of the net littoral transport patterns on wind data. Data from wind roses at Caplen, Brownsville and Corpus Christi were converted into wave parameters that were used to hindcast resulting wave conditions. A wave height-to-period relationship was developed and wave refraction corrections were applied. A total wave work equation was then applied to compute the frequency of occurrence-weighted littoral transport rates. From this they stated, "By similar procedure at Corpus Christi, the estimated annual northward littoral transport is 142,000 cu yd at 6 ft depth, and 112,000 cu yd southward at 6 ft depth, for a total of 254,000 cu yd. Subject to modification by hurricane waves and Gulf currents, it appears that normal wave action shifts the littoral transport almost equally northward and southward at Corpus Christi . . . "(op cit. P.241). Note that this study estimates a net up-coast transport of about 30,000 cu yds/yr.

Watson and Behrens (1976) used data from a USACE Coastal Engineering Reseach Center program of systematic daily visual wave observations (LEO) to compute net and gross littoral drift rates. The observations were made 1 mile southwest of Aransas Pass and consisted of significant wave height, wave period, angle between the shore and the waves, wave breaker type, longshore current speed and direction and wind speed and direction. The data represented the period between September 1972 and June 1975. They computed the daily littoral transports and averaged them. The resulting mean annual gross transport was 726,200 cu yd and the corresponding net value was 66,600 cu yd or about 8% of the gross. The net over this 37-month period was down-coast (southwestward). This location is 12 miles northeast (up-coast) of Packery Channel (then called Corpus Christi Pass). From the pre-construction period (approximately spring of 1972 as the inlet was completed in August 1972) to March 1975 there was net deposition (up to ~2,000 ft^3/ft) on the southwest side of the jetties and net erosion (up to ~ 1,400 ft^3/ft) northeast of the jetties. This would indicate net up-coast transport at the pass (the location of the proposed re-opened Packery Channel).

There is not a consensus that the geological evidence supports the concept that there is a time-averaged net down-coast (southwestward) littoral transport at the site of Packery channel. From the forgoing summaries of individual studies it can be seen that there are trends in some of the geological parameters that are meaningful in developing an understanding of littoral transport patterns on the coastal engineering time scales (i.e. decades not millennia). The patterns of distribution of heavy minerals and shells show a net northward transport of sediment northward to the barrier island adjacent to Baffin

Bay. On the other hand, the analyses of wind and wave-driven littoral transport carried out by Watson (1971), Price (1932) and Carothers and Innis (1962) all suggest that a time-averaged convergence should be located in the vicinity of Port Aransas.

In interpreting the geological data it is important to understand the dstinction between the effects of net advection and time-averaged diffusion on the distribution of minerals, shells and sediment grain size parameters. With rare exception littoral sand transport is characterized by a relatively small net transport in the presence of much larger back and forth excursions. These up- and down-coast variations occur on time scales of hours to days. In some places there is no net transport at all. Where net transport does occur it can be thought of as advection in a time-averaged sense.

The distribution of geological tracers such as heavy minerals and shells results from the combined effects of advection and diffusion. If there is a clear net transport then a tracer is carried along and streaked-out from its source. However, a similar distribution pattern results when there is no net transport, just a sequence of up-coast and down-coast transport events. Tracer grains picked up at a source must also move back and forth so that, over time, these grains work themselves long distances from where they entered the littoral transport system. If there is a sink for the sediment in the form of beach deposits or deposits formed inland due to wind-transport from the beach, then there will be a concentration gradient decreasing away from the source along the beach.

With these processes in mind it is possible to explain many of the observations in the geological studies. The combined effects of advection and large-scale diffusion in the littoral transport cause the patterns of diminishing heavy mineral concentrations and whole shells leading northward from the Rio Grande. Both of these act in the same direction. It appears that between Aransas Pass and the beach adjacent to Baffin Bay the effects of net transport (time-averaged advection) are overridden by large-scale diffusion. The sharp gradients in heavy mineral suites, grain size distributions and shell content noted to occur in a zone 15 to 40 miles southwest of Packery channel are brought about by the large-scale diffusion of sand from more northerly sources meeting the flux of sediment moving northward from the Rio Grande. If there is a convergence of time-averaged littoral transport on the Texas coast it is not fixed in location of the time scales of importance to coastal engineering (decades not millennium). Instead, the location of time-averaged littoral drift convergence migrates up, and down the coast. Its limits are probably described by Gulf beaches near Rockport and Baffin Bay. The area in between these limits is dominated by patterns of back-, and forth littoral transport (called gross or large-scale diffusive transport) with annual net transports that can be direct either up or down the coast.

The review of geological evidence and the analysis of the 'updated' WIS hindcast and Corpus Christi Naval Air Station (NAS) wind data indicate that the conflicting conclusions cannot be adequately resolved and in fact the net littoral transport direction may be subject to long-term variations ranging from north to south directed transport. We, therefore, have adopted the approach that net littoral transport may be from the north or the south for any extended period of time (decades) and have conducted the sediment transport and shoreline evolution studies to reflect this possibility. The details of the studies and the representation of both north and south transport directions are given in Section 5.

4.2 ANALYSIS OF WAVE VARIABILITY

The various wave data sets described in Section 3 have durations ranging from 3 to 20 years. There is a question of how representative of long-term conditions the time span of any of the data sets are given that wave conditions are known to exhibit considerable inter-annual variability. This variability can be expressed in patterns of several to many consecutive years of more or less energetic conditions. The directions that the waves approach the shoreline also exhibit long-term patterns of variability. The major application for the selected wave data set is in the modeling of a representative littoral drift volume flux. In analyzing littoral drift rates near the proposed Packery Channel, it is especially important to consider the long-term variations in the breaker angle, because this shore is located near a place where the up-coast and down-coast transports converge. Patterns of inter-annual variability can cause the location of the convergence to shift many tens of miles along the coast.

To understand the patterns of long-term variability of waves it is desirable to have a record longer than any of the available wave data sets. It was necessary to adopt 40 years of wind data from the Corpus Christi Naval Air Station to obtain information about weather pattern variations spanning multiple decades.

Because our interest is in developing estimates of the long-term variations in the rates and directions of the littoral drift, we need to only consider the relative differences in transport magnitude. These rates can be approximated in a screening analysis, similar to that used by Carothers and Innis (1962), by using standard expressions for predicting locally generated open sea waves which relate the wave height and period to the wind speed (for example, see USACE Shore Protection Manual, 1984, Chapter 3). For the moment the interest is in the relative patterns of long-term wave-driven littoral drift, so the wave height and period relations are given as:

$$H_{rel} = K_1 (U^2)$$

and ,

$$T = K_2 (U).$$

Here H_{rel} is the relative wave height, T is the corresponding wave period, U is the wind speed, and K_1 and K_2 are proportionality constants. In our screening analyses, the relative wave height and corresponding period were calculated every three hours over the 40-year wind record. The offshore, locally generated wave direction was taken to be the wind direction.

The main use of the wave periods was to assist in adjusting the wave data from locally generated offshore conditions to conditions at the breaker line through an application of Snell's law for wave refraction. This resulted in a computed breaker angle corresponding to each wave height in the eight daily entries in the 40-year record. Noting that only the relative temporal patterns of wave height are of interest at this stage we ignored direct shoaling effects. The estimated wave height and period were transformed to a littoral drift rate using an equation by Komar (1976) which has been developed from radiation stress theory to relate the littoral drift volume flux to the wave breaker height and breaker angle:

$$Q_{long} = K_3 (H_{rel})^{5/2} (\sin \alpha_b \cos \alpha_b)$$

Where α_b is the breaker angle and K_3 is a proportionality constant.

The 40-year airport wind record was converted to a corresponding time series of relative littoral drift fluxes using these expressions. Clearly, the method is an approximation of actual wave conditions. Aside from limitations of the wave prediction equations, only offshore locally generated waves are considered. This ignores waves propagating to the site from farther away, and these waves commonly arrive as long period swells. Also, fully developed wave conditions result from the calculation while, in actuality, the winds are often changing in time. Furthermore, the wind record used was taken onshore and relatively close to the beach. This means that the record contains the effects of the local sea breeze - land breeze diurnal cycle. The corresponding winds are known to die-off seaward of the shoreline, so that fully developed waves are limited by the available fetch. However, while recognizing the limitations of the derived record, there is such a strong correlation between wind and wave conditions that useful information about the long-term patterns of littoral drift variations are still exhibited.

Figure 4-1 shows two segments of the 40-year estimate littoral drift flux record to illustrate the relationship between the original 3-hourly data and the same data subject to a weekly time-average. The fundamental pattern of variability is apparent in both records. Figure 4-2 shows the full 40-year record with the periods of available wave data indicated as time lines. This shows that there is a noticeable pattern of inter-annual variability in both the rate and direction of littoral drift. Figure 4-2 also shows that the periods of available wave data are representative of different portions of the overall pattern of variations. This point will be expanded upon later in this report where the periods of modeled conditions to average and extreme conditions are related.

5.0 LITTORAL TRANSPORT MODELING

5.1 MODELING APPROACH

Sediment transport and shoreline evolution modeling has been employed to help quantify the impacts of the jetty construction on shoreline position, sand transport and channel dredging. The USACE GENESIS computer model (NEMOS software package ,Veri-Tech, Inc.) was used throughout the study. Prior to applying the model to predict conditions at the proposed Packery Channel, the model was calibrated to conditions at Mansfield Pass. Mansfield Pass, located about 75 miles south of the proposed location of the re-established Packery Channel, provides a good analog that can be used for model calibration. It is of similar size and orientation relative to the shoreline as the proposed Packery Channel breakwaters (jetties) and the construction, shoreline evolution, channel sedimentation, and dredging history between 1962 and 2000 is documented.

After the successful calibration to conditions at Mansfield Pass, the model was employed to predict conditions for the proposed Packery Channel. As noted earlier, the 'updated' WIS hindcast data for station #1087 (near Packery Channel) predicts a predominant net longshore transport to the North. In order to consider the possibility of prolonged periods of net transport to the south, and to access the impacts of the jetties for a southward transport, we have conducted a second modeling analysis based on rotating the WIS data by 10 degrees. In the modeling analysis, scenarios based on both the un-rotated and rotated wave data were used.

For all of the modeling analysis, including the calibration, the offshore WIS wave data were transformed to nearshore conditions using the STWAVE model. All named cyclonic storm events within 200 miles of the sites (Mansfield Pass or Packery Channel) with wave heights above 2.9 m were removed from the 'updated' WIS data sets and replaced with average conditions. These storms can have unpredictable results on the sediment transport at the sites and the GENESIS model does represent the effects of large localized currents, wind/wave set-up and other effects associated with the storms. The storms removed from the wave data sets for each site are shown in Tables 5.1 and 5.2.

Table 5.1 Storm Events Removed from Mansfield Pass Wave Records

Category	Name	Start Date	End Date	Distance from Mansfield Pass (miles)	Direction from Mansfield Pass	Maximum Wave Height (meters)
H	Allen	7/31/80	8/11/80	20	S	7.8
TS	Amelia	7/30/78	8/1/78	20	S	3.1
H	Barry	8/23/83	8/29/83	50	S	5.2
TS	Arlene	6/18/93	6/21/93	50	N	5.1
H	Anita	8/29/77	9/3/77	100	S	7.7
H	Gilbert	9/8/88	9/20/88	100	S	7.7
H	Jeanne	11/7/80	11/16/80	150	E	5.4
H	Alicia	8/15/83	8/21/83	150	NE	2.9
H	Jerry	10/12/89	10/16/89	200	E	3.3

Table 5.2 Storm Events Removed From Packery Channel Wave Records

Category	Name	Start Date	End Date	Distance from Packery Channel (miles)	Direction from Packery Channel	Maximum Wave Height (meters)
H	Allen	7/31/80	8/11/80	90	S	2.9
TS	Amelia	7/30/78	8/1/78	90	S	2.5
H	Barry	8/23/83	8/29/83	120	S	3.7
TS	Arlene	6/18/93	6/21/93	20	S	5.7
H	Anita	8/29/77	9/3/77	170	S	6.2
H	Gilbert	9/8/88	9/20/88	170	S	6.1
H	Jeanne	11/7/80	11/16/80	150	E	3.9
H	Alicia	8/15/83	8/21/83	150	NE	2.3
H	Jerry	10/12/89	10/16/89	200	E	3.0

5.2 CALIBRATION TO HISTORICAL CONDITIONS AT MANSFIELD PASS

Data available for calibrating the model to conditions at Mansfield Pass span the time period from 1957 through the year 2000. However, certain data are only available for shorter periods, and conditions at the pass changed over time, dictating the need to select the appropriate calibration period and data sets. Data for the earlier periods (1957 – 1973) are documented by Kieslich (1977). The original jetties were constructed of tetrapods, which, soon after construction, began sinking into the sediments and became ineffective. By 1962, the current jetty system was completed. The present jetty spacing is 1,000 ft and the new jetties extend approximately 1700 ft from the 1957 shoreline position. The jetty alignment is nearly due east which orients the jetties approximately 22 degrees clockwise off the shore normal direction. By 1962, the shoreline on the South side of the jetties had already built out substantially, due in part to the presence of the tetrapod jetties. Maps of detailed shoreline position are available in Kieslich (1977) and more recent positions of the shoreline to the south of the jetties has been obtained from 1974, 1993, and 1999 aerial photographs obtained from the Texas Natural Resource Division. A time series of shoreline positions at the south jetty is plotted in Figure 5-1. The shoreline built out steadily until the early 1970s, and then the rate of build-out slowed, and the shoreline asymptotically approaches a location that is few hundred feet from the south jetty tip. The shoreline to the north of the north jetty generally is eroding and retreating from the 1957 position. It is apparent from the shoreline migration records that there is a significant net transport to the north.

Dredging records are available for the Galveston District of the USACE starting from 1962 through present day. These records, which were summarized in Kraus and Heilman, (1997), were reviewed and updated to include recent dredging records from 1996 to 2000. The channel was originally dredged to 25 ft, which required the channel to extend about 700 feet offshore from the tips of the jetties. From 1982 onward, the channel depth was maintained at 16 feet. Figure 5-2 shows the cumulative dredging volume over the time of record. The slope of the cumulative curve at any time represents an estimate of the dredging rate. The average annual dredging requirements for the entire period, 1962-2000, is 160,000 cubic yards per year (cy/yr). However, the curve indicates that the dredging requirements varied over time. From 1962 to 1972, the rate averages about 102,000 cy/yr. After that the rate becomes much higher, 275,000 cy/yr until about 1982, after which the rate averages about 99,000 cy/yr. The dredging rates from 1962 to 1982 show a general increase which, in part, is associated with the south shoreline build out. There were a number of storms to the south of Mansfield Pass during the 1970's, which could also account for the increased dredging during that period. The change in the dredging rate from 275,000 cy/yr to 99,000 cy/yr corresponds to the change in the dredging depth of the channel and may be associated with the reduced capacity of the Mansfield Channel to capture sediment.

Based on the data review, it is feasible to create a 38-year period, 1962 through 2000, for model calibration at Mansfield Pass. The primary issue with this period is the availability of wave data to use as input to the model. The nearby 'updated' WIS station, #1092, only covers the period from 1975 through 1996. It is possible to restrict the calibration to the 75-96 period. However, it is recognized that in the applications of the model to Packery Channel, the initial shoreline will be far from the jetty tips and it would be beneficial to calibrate the model at Mansfield Pass for similar conditions (i.e. 1962-1975). We considered using the old WIS hindcast data (1956-1975) but unfortunately, the hindcast data are not considered to be of good quality. Therefore, we used the 'updated' WIS data (with cyclonic storms removed) to cover the earlier period under the assumption that the general wave patterns should be similar between both periods. Since we are only attempting to calibrate to the long-term trends in shoreline migration, and not detailed year to year variations, this approach is valid.

The model calibration was conducted by adjusting the model parameters to produce a shoreline migration similar to that reported for the period from 1962 to 1999. An initial shoreline position corresponding to 1962 was used to begin the model simulations. Historical records of shoreline movement indicate that the shoreline in the vicinity of Mansfield Pass (outside of the effects of the updrift/downdrift accretion and erosion) is stable (Paine and Morton, 1982), and therefore no background recession rate was applied in the model. A series of successive simulations were made, each with a revised set of model parameters, until the predicted shoreline evolution was in agreement with the measured positions for the shoreline south of the south jetty. A good fit to the measured data was obtained using transport coefficients of 0.55 for K'_1 and 0.25 for K'_2, 15 ft for the depth of closure, and 0.05 for the permeability coefficient. The predicted shoreline position, relative to the tip of the jetty is plotted in Figure 5-3. In general, the agreement with the measured data is quite good, indicating that the model parameters and the 'updated' WIS data are valid for the southwestern portion of the Texas coast.

With the model calibrated, estimates of the longshore transport could be made for the vicinity of Mansfield Pass (in the absence of the jetties). The average annual gross transport is 503,000 cy/yr with a northward-directed net transport of about 319,000 cy/yr. The model predicted the average transport around the south jetty tip (to the north) to be 158,000 cy/yr, which is consistent with the long term average dredging requirements at Mansfield pass. A comparison of the measured dredging volume and predicted jetty tip bypassing is shown in Figure 5.4 for the period after 1982. Also shown is an adjustment of the predicted tip bypassing by a trapping coefficient. The model predicts that 205,000 cy/yr may move around the jetty tips, while dredging of 99,000 cy/yr was required at the pass. The ratio of measured dredged sand to predicted tip bypassing is approximately one half. This ratio does not necessarily imply a model over-prediction of dredging requirements, because it is likely that some of the littoral drift sand is naturally bypassing the jetty tips. The trapping coefficient used in Figure 5.4 represents this natural bypassing process.

In the earlier period, from 1962-1973, the predicted tip bypassing rate was typically about 73,000 cy/yr, and from 1973 onward it was approximately 205,000 cy/yr. In comparison to the measured dredging rates for the earlier period, the model predicts less sand to be moving around the jetty tip than was dredged during the same time period. However, the dredged channel extended about 700 ft beyond the tip jetties during this period. The model does not account for other offshore sources of sand that may contribute to shoaling in this portion of the Mansfield Channel, and thus a direct comparison during this period is not appropriate.

5.3 APPLICATIONS TO PACKERY CHANNEL

The calibrated GENESIS model has been configured for the Packery Channel shoreline in order to access the impact of the jetties on the shoreline migration and to develop requirements for sand management. Nine modeling scenarios were developed and were applied for both the original 'updated' WIS data and the rotated (10 degrees) 'updated' WIS data for a total of 18 scenarios. Note that the results of some of the scenarios required numerous iterative simulations to determine the appropriate mechanical by-passing rates to control the shoreline evolution. The 9 cases are listed in Table 5.3.

Table 5.3 Model Scenarios

Scenario	Purpose	Description
1	Establish Baseline	Simulate sand transport with jetties and no sand management
2	Sand Management	Establish optimal mechanical by-pass rate to maintain shoreline position
3	Sand Management	Simulate effects of bypassing on annual basis
4	Sand Management	Simulate effects of bypassing on biennial basis
5	Dredge Material Placement	Effect of distributing dredge material (sand) over a short length (on down drift side)
6	Dredge Material Placement	Effect of distributing dredge material (sand) over a long length (on down drift side)
7	Initial Beach Fill	Effect of placing fill on south side only
8	Initial Beach Fill	Effect of placing fill on both north and south sides
9	Jetty length	Determine effects of jetty length on sand management

The shoreline in the vicinity of Packery Channel has been eroding at an average rate of 5 ft/yr, and is attributed to wind blown sand losses (Kraus and Heilman, 1996). In the modeling analysis, it is necessary to reflect this loss of sand from the littoral system in order to properly model future shoreline evolution. A cell budget has been constructed that is based on modifications to the budget developed in the previous report (Kraus and Heilman, 1996). The previous budget had the following transport rates:

Table 5.4 Original Cell Budget

Source	Rate (cu yds/yr/ft)
Littoral transport accumulation	0.2
+ Loss due to wind blown sand	-3.2
= Net erosion of shoreline	-3.0

The 3.2 cu yds/yr/ft loss rate was determined by balancing the calculated littoral accumulation obtain from the model simulations (0.2 cy/ft/yr) with the measured shoreline recession (-3.0).

We have used the calibrated GENESIS model to obtain revised estimates of the littoral transport accumulation in the vicinity of Packery Channel using the 'updated' WIS data as input. The initial shoreline in the modeling analysis was obtained from NOAA Chart #11307. The calculated net littoral transport was estimated to be larger than that obtained in the previous anlaysis (Kraus and Heilman, 1996), approximately 60,000 cy/yr over a 36,000 ft reach centered on the proposed Packery Channel location. The associated littoral transport accumulation is 1.6 cy/ft/yr (compared to 0.2 cy/ft/yr). The revised cell budget is shown in Table 5.5.

Table 5.5 Revised Cell Budget

Source	Rate (cu yds/yr/ft)
Littoral transport accumulation	1.7
+ Loss due to wind blown sand	-4.9
= Net erosion of shoreline	-3.0

The sand loss rate of –4.9 cy/ft/yr was used in all modeling scenarios to account for wind blown sand losses. This higher value is still well within the upper bound for wind blown sand losses derived from measurements of 21.2 cy/ft/yr (Kraus and Heilman, 1996). Similar values were obtained for the rotated WIS data scenarios and used in those simulations.

5.4 LONGSHORE TRANSPORT RATES FOR PACKERY CHANNEL

Estimates of the longshore transport have been made using the calibrated GENESIS model for the shoreline in the vicinity of Packery Channel in the absence of the proposed jetties. The method is similar to employing the SEDTRANS model. The estimated average gross and net transport for the 1976 through 1995 period are 365,000 cy/yr and 188,000 cy/yr to the north, respectively. These values are about 60% of the rates obtained at Mansfield Pass. The smaller rates are due to smaller wave heights and periods at Packery Channel than at Mansfield Pass, and also due to differences offshore incident wave angles. At Mansfield Pass, the incident wave angle is about 40 degrees, whereas at Packery Channel, it is on the order of 5 to 10 degrees. Table 5.6 shows the calculated gross and net at the proposed Packery Channel site for each of the simulated years.

When the 'updated' WIS data is rotated 10 degrees, the annual average gross and net are 270,000 cy/yr and 170,000 cy/yr with the net transport directed to the south. These values are comparable to the un-rotated WIS results, except for direction, and also have gross-to-net ratio that is similar to that of the OCTI hindcast-based gross-to-net transport ratio. It is noted that the rotated 'updated' WIS data yield higher transport rates compared to the OCTI based results, but the gross-to-net ratio is similar. We did experiment with other rotations of the 'updated' WIS hindcast data, including angles of 8 and 12 degrees, and found that the 10 degree rotation was most consistent in gross-to-net ratios to both the OCTI hindcast-based results and the un-rotated 'updated' WIS data.

The net transport rates of 188,000 cy/yr (or 170,000 cy/yr to the south) form the basis for required annual average mechanical bypassing rates. However, these values need to be modified to account for natural bypassing around the jetty tips, which is discussed in section 5.5.

5.5 EVALUATION OF SHORELINE CHANGES AFTER JETTY CONSTRUCTION

The calibrated GENESIS model has been used to explore the effects of the proposed jetty design on sediment transport in the vicinity of Packery Channel. Both the original 'updated' WIS data and the rotated WIS data have been used as input to the model for each of the nine scenarios described in section 5.3.

Table 5.6 Estimated Gross and Net Transport at Packery Channel

Year	Gross (x10^3 cy/yr)	Net (x10^3 cy/yr)
1976	442	190
1977	293	40
1978	273	125
1979	364	115
1980	312	130
1981	332	120
1982	351	125
1983	403	120
1984	442	205
1985	416	125
1986	312	152.5
1987	312	125
1988	357	132.5
1989	429	250
1990	403	120
1991	286	125
1992	403	230
1993	390	185
1994	416	150

Shoreline data obtained from NOAA Chart #11307 was used for the initial shoreline in each simulation. The approximate shoreline azimuth at Packery Channel was estimated as 207 degrees. The proposed Packery Channel jetties will extend 1,400 ft from shore. The jetties will be skewed 12 degrees north of shore normal to match the existing channel alignment. The jetty skew was incorporated into the model. The seawall, located to the south of the proposed south jetty, was also included. Each model scenario started with the present shoreline, and consisted of a 20-year simulation to estimate future impacts.

The initial and final shorelines for the baseline case (scenario 1, original WIS data) are shown in Figure 5.5. The development of a fillet south of the south jetty, due to the net transport to the north, is evident. Subsequent erosion is predicted to the north of the jetty system, on the order of 350 feet adjacent to the north jetty over the 20 year period. The influence of the background shoreline recession rate is most evident at the south boundary, where the shoreline is predicted to recess about 100 ft.

The fillet predicted for the Packery Channel jetties is longer than that developed in the calibration to Mansfield Pass. Fillets tend to orient their shoreline normal to the average incident wave energy direction (personal communication, Dr. Robert Dean). At Mansfield, the incident direction is from almost 45 degrees to shore normal, causing the fillet shoreline azimuth to turn more towards shore normal than at Packery Channel, where the incident wave energy is only about 5 or 10 degrees from shore normal. In this scenario, the seawall is well protected by the shoreline advancement associated with the fillet.

The amount of transport predicted to cross the jetty tip (i.e. gross transport) increases over the twenty-year period, as the shoreline builds out along the south jetty. The natural tip bypassing predicted by the model is plotted in Figure 5.6 with a linear regression to quantify the trend. Base on the trend line, the transport rate increases from about 75,000 cy/yr initially to about 95,000 cy/yr after 20 years. Over this period the shoreline has built out to a point 1000 feet from the south jetty tip.

The next scenario examined (scenario 2, original WIS data) represents conditions if optimal mechanical bypassing was used to maintain as close to as possible, the original shoreline position. The mechanical bypassing rate applied is based subtracting the natural bypassing from the annual average net transport obtained in the absence of the jetties (i.e. ~ 188,000 cy/yr). The natural bypassing rate is taken to be that which occurs around the jetty tips when the shoreline is near its initial position The average natural bypassing rate, based on the model simulations, is 38,000 and consists of 28,000 cy/yr from the south and 10,000 cy/yr from the north. The natural bypassing rate from the south (28,000 cy/yr) was subtracted from the net transport to obtain an estimate of the required annual average mechanical bypassing which is 160,000 cy/yr. This approach assumes that the bypassing is continuous and maintains the original shoreline position. The value of 160,000 cy/yr, was found to yield satisfactory results in modeling simulations. The results after the twenty-year simulation are shown in Figure 5.7. It is clear that the bypassing is sufficient to maintain the original shoreline. These results confirm that the annual-average required mechanical bypassing at Packery Channel, in the absence of cyclonic storms, is approximately 160,000 cy/yr. In order to determine the variability in the required bypassing rate, a cumulative frequency curve was developed based on the model result and is shown in Figure 5.8. The results indicate that fifty percent of the time, the required dredging will be about 140,000 cy/yr. This is less than the annual average rate (i.e. mean) because the median is skewed upward by a few larger events.

The natural bypassing (transport past the jetties) calculated by the model averages approximately 38,000 cy/yr (when the initial shoreline position is well maintained by continuous sand management). The annual variability is demonstrated in Figure 5.9, which shows the predicted natural bypassing for each year. In developing an estimate of dredging rates for Packery Channel it is important to consider both the modeling results (natural tip bypassing) and values obtained from the dredging records and Mansfield Pass. Important considerations in the comparison include the following:

- Wave energy and gross transport at the Packery Channel site are smaller than corresponding values at Mansfield Pass;

- Packery Channel is predicted to have similar stability characteristics or marginally better characteristics than the channel at Mansfield Pass (see Section 8.0 for discussion of channel stability characteristics);

- The Mansfield Pass channel has more capacity to store sediment (wider, deeper and longer);

- Mansfield Pass was dredged beyond the jetty tips, Packery Channel will not be dredged a significant distance beyond the jetty tips; and

- The proposed Packery Channel jetties are shorter than those at Mansfield Pass.

The modeling analysis at Mansfield Pass (1982-1999) indicates higher predicted natural bypassing of the south jetty than was needed to explain the channel dredging record. It is likely that the difference is due to some bypassing of sand past both jetties to the downdrift side. The ratio of the measured dredged sand to the predicted bypassed sand was about 1/2. If this ratio was applied directly to the model predictions of jetty bypassing and Packery Channel, the predicted dredging requirements (annual average) at Packery Channel would be approximately ½ of the model-based natural bypass amount. However, the predicted natural bypassing for Packery channel is substantially lower than that at Mansfield Pass (38,000 cy/yr compared to 205,000 cy/yr) and it is likely that the proposed Packery Channel could trap all of the sand predicted to pass the jetties (i.e. a trapping efficiency of 1). Therefore, as a conservative estimate of the dredging requirements at Mansfield Pass, the model-based values of tip bypassing are used without any alterations as an estimate of the dredging requirements at Packery Channel. Using this approach, the annual average channel dredging requirements at Packery Channel is estimated to be 38,000 cy/yr. To assess the variability in the annual channel dredging requirements, a cumulative frequency curve has been generated, much like that used for the annual bypassing requirements. The curve is shown in Figure 5.10.

The capacity of the proposed channel (within the extent of the jetties) is estimated to be about 98,000 cy. This estimate is based on a nominal width of 194 ft, a length of 1400 ft and depth of 12 ft. The depth is based on the difference between the depth of 14 ft (authorized depth of 12 ft plus 2.0 ft of allowable over-depth) and the 2 ft shelf flanking the channel. The channel navigation will be compromised long before the capacity is filled, indicating that dredging of the channel may be required on at least an annual basis.

The two scenarios 3 and 4 (original un-rotated WIS) are intended to determine if multiple year bypassing is feasible. For scenario three, the bypassing was completed over a one month time period at the end of each of the 20 years of simulation. For scenario 4, twice the amount, 300,000 cy was bypassed over a one month period at the end of every second year. Figure 5.11 shows the results for scenario 4, which occur soon after the end of the last bypassing period. The slight build-out of the north shoreline is evident due to the recent distribution of bypassed sand. In scenario 4, the shoreline along the south jetty did build out before each bypassing operation, causing more sand to pass around the jetty tip. The increased amount was approximately 50,000 cy/yr and should be viewed as a potential increase in channel dredging requirements.

Application of scenarios 5 and 6 (original un-rotated WIS) are intended to investigate the effects of distributing the bypassed and channel dredge material along the shoreline. It is assumed that all of the channel dredged material is placed on the downdrift side (north for these cases). In scenario 5, the dredged material was distributed over 500 ft extending north from the north jetty. In scenario 6, it was distributed over 3000 ft. In both cases the effects on the shoreline were very temporary and the model predicted little sensitivity to the dredge material placement on shoreline evolution.

Scenarios 7 and 8 (original un-rotated WIS) were used to study the effects of placement of the initial beach fill material made available during the channel construction. Approximately 700,000 cy of sediment is available for initial placement. We have considered two cases. In scenario 7, the beach fill was all placed to the south to provide initial protection of the seawall. The model predicted the material to provide additional protection for about 3 years, after which the shoreline returned to its original position. The beach fill material was dispersed both to the north and south, but the northward movement was limited due to the jetties. The amount that passes the jetties to the north is slightly increased due to the initial shoreline build-out associated with the beach fill. In scenario 8, the beach fill was divided equally between the north and south jetties and placed over a distance of 4400 ft along the shoreline from each jetty. The predicted shoreline response indicates that the fill will have an impact for approximately 3 years, after which the shoreline will return to its initial position.

In scenario 9 we have considered the effects of extending the jetties approximately 150 ft. The 150 ft distance is that which could be obtained using the efforts that would otherwise have been applied toward the construction of the spurs.

The model predictions indicate that the jetty extension would reduce the channel dredging requirements by less than 1000 cy/yr. This low sensitivity is due to the relatively long distance between the shoreline and the jetty tips maintained by mechanical bypassing.

The simulations for all 9 scenarios using the rotated WIS data have also been completed. For the purposes of estimating the gross and net transport, the required mechanical bypassing and the channel dredging, the results obtained using the rotated WIS data essentially yielded nearly the same results obtained with the un-rotated WIS data. This occurs because only the magnitude of the gross and net transport, and not the direction of net transport is important for these calculations. Since the model-based gross and net transports had approximately the same magnitude for both the un-rotated and rotated WIS wave data sets, the estimated mechanical bypassing and channel dredging requirements are the same in both scenarios. However, the net transport direction is important from a shoreline evolution standpoint, primarily due to the presence of the seawall to the south of the proposed jetty location. The relevant modeling results effecting shoreline position are discussed below.

The two most revealing scenarios for the rotated WIS data cases are the baseline case and the one and two year bypassing schedules. Figure 5.12 shows the final shoreline position after 20 years if no sand management is used (only channel dredging). Clearly the erosional pattern to the south causes the shoreline retreat to impinge upon the seawall. Thus, if the net transport is predominately from the north, active and aggressive sand management will be required to keep the downdrift beach (now to the south) from eroding. We have also examined the results of annual and biennial sand bypassing, as in the previous un-rotated cases to see if, over the twenty-year period, the seawall would be exposed. This may occur if successive years of southward transport created substantial downdrift erosion before the scheduled bypassing occurred. However, based on the model results, the width of the original beach was essentially maintained for both annual and biennial bypassing schedules.

5.6 SUMMARY OF LITTORAL TRANSPORT MODELING RESULTS

The littoral transport modeling study is based on the GENESIS model and has been successfully calibrated to historical conditions at nearby Mansfield Pass. The calibrated model has been used to estimate the effects of the proposed Packery Channel jetties will have on shoreline migration, required sand management and channel dredging.

The sand management options for Packery Channel are difficult to develop because the direction of sand transport is not well defined. The proposed Packery Channel lies near a nodal point in the sand transport system, and there is conflicting evidence as to the long-term net transport direction. The annual average required mechanical bypassing of sand to maintain the current shoreline position is 160,000 cy/yr. The variability about the annual average is expressed in a cumulative frequency chart (Figure 5.8). We expect that there will be extended periods, lasting one or more years, where there will be alternating up- and down-coast net sand transport.

During periods when the net transport direction proves to be to the north, the seawall will be naturally protected by the generation of a fillet along the south jetty. It may be possible to let the fillet build out to a prescribed location before sand management is invoked, thus providing additional protection for the seawall. However, this approach will be at the expense of some erosion on the north beach. During periods when the net transport direction is to the south, more aggressive sand management will be required to keep the downdrift erosion from encroaching on the seawall.

It is recommended that a monitoring program, consisting of regular beach profiling in the vicinity of the jetties, be maintained to provide data on the net transport direction and the need for sand bypassing. Also, monitoring after major storms should be conducted.

Monitoring should be done seasonally, and after major storms initially. The monitoring interval can be lengthened but should be at the least, annual It is also recommended that any pre-construction sand bypassing configurations be designed to accommodate bi-directional sand bypassing.

The estimated channel dredging is 38,000 cy/yr and the variability is shown as a cumulative frequency chart (Figure 5.10). This estimate assumes that wind blown sand does not contribute to the dredging requirements and does not include potential channel shoaling due to cyclonic storms. It is recommended that the channel bathymetry be monitored on a regular basis and after major storm events so that dredging is scheduled before shoaling becomes a navigation hazard.

Because the direction of the net littoral drift transport cannot be reliably predicted, the placement of the beach fill material, obtained from the initial construction dredging, is not obvious. Model analysis indicates that, for reasonable fill lengths along the beach, the effects of the beach fill will last about 3 years after which the shoreline tends to return to its original position. There is interest in placing the material in front of the seawall to provide additional protection from major storm events. If the long-term net transport is to the north, then this would only have been useful to provide initial protection, since the development of an updrift fillet would provide similar protection. However, if the transport is found to be from the south, the fill will provide protection that would otherwise not be available. Two options for placement are recommended. The first places the sand on the south side, in front of the seawall. The second option places the sand, equally to the north and south of the jetty.

Under normal conditions, sand bypassing can be conducted on either continuous, yearly or biennial schedules without significant impacts to the downdrift beach. However, channel dredging will likely be required on an annual basis, because the estimated sedimentation rate in the channel could fill the channel within one year.

Adequate maintenance dredging and sand bypassing combined with a rigorous beach and channel monitoring program can minimize beach erosion in the vicinity of the channel.

The effects of significant tropical storms, hurricanes or other extreme events are not included in the modeling analysis and predictions. These events could significantly impact channel siltation resulting in emergency dredging and beach erosion.

6.0 APPRAISAL OF LARGE-SCALE VARIABILITY

The modeling results given above reflect the application of the wave data from the 'updated' WIS Station #1087 for the 20-year interval between 1976 and 1995. In an earlier section of this report it was noted that the wave climate exhibits variability over a range of time periods. We now turn our attention to applying the results of the model runs representative of a particular 20-year time interval to estimates of the channel infill rates during periods more representative of long-term average conditions. We also consider the possibility of a sequence of several adverse years leading to higher than average infilling rates. Channel sediment during these periods is evaluated so that realistic plans can be made to provide an adequate dredging schedule to keep the channel open during these times.

Figure 4-2 shows the 40-year time series of estimated littoral drift flux at Packery Channel as it has been estimated from a corresponding wind record. The previously noted annual pattern (Weise and White, 1980) favoring up-coast (northeastward) transport during winters and down-coast (southwestward) transport during the summer is generally apparent. It is also clear that there is considerable variability in the relative intensities of these patterns when year-to-year comparisons are made. It is therefore reasonable to identify a year as a reasonable time interval to quantify in making statements about the fundamental patterns of long-term variability. The annual average root-mean-square (RMS) of the transport fluxes provides a simple and meaningful measure of the littoral drift behavior.

Table 6.1 shows a tabulation of the annual average RMS values for the 40-year record shown on Figure 4-2**.** The annual total RMS value is a reasonable proportional approximation of the total flux of littoral drift at the mouth of Packery Channel that leads to channel sedimentation and filling. That is, the channel will undergo more sedimentation in years when the annual total-RMS littoral drift flux is large. Although antecedent shoreline shape and position is important, in the following analyses we assume that this proportionality between the measure of the relative littoral drift flux and the rate of channel sedimentation holds for all values in Table 6.1. This is used to estimate the inter-annual variability of channel sedimentation rates.

It is also noted that the difference between the up-coast and down-coast values of the annual-average littoral drift RMS is proportional to the net annual littoral value. Table 6.1 shows these values. This metric also exhibits considerable inter-annual variability and patterns of sequential years with similar trends. In fact, there is a dominance of up-coast transport over the 40-year record. This suggests that if there is a "null-point" in the large-scale coastal littoral drift pattern of the central Texas coast it shifts position to either side of the Packery Channel, and it was located up-coast of Packery Channel during most of the time covered by the 40-year record.

The minimum value of the annual average littoral drift RMS is 393, occurring in 1974 and the maximum is 1964, occurring in 1960. Remember, the values are meaningful only in a relative sense. The annual maximum is 244% of the long-term mean annual average, and the annual minimum is a corresponding 51% of the mean average. It should also be noted that the occurrences of high and low values tend to occur in series of sequential years, and this has particular relevance to dredging and sand management planning.

The annual average littoral drift RMS values can also be used to explore the differences in the results of applying the different wave-data sets to the modeling of sediment transport in the project area. It has been noted that the results of the Blucher Institute study (Kraus and Heilman 1997) appear to be lower than might be expected with alternate methods of evaluation (Hayes, van de Kreeke and Dean, 1997). The year-to-year variability of the littoral drift rates is shown on Figure 6-1, where each year is represented by its annual average RMS value. Reference to Figure 6-1 and Table 6.1 shows that the seven years of OCTI wave hindcast data represent a sequence of low annual average RMS values in the overall 40-year record. The average over these seven years is 84 % of the overall average for the 40-year record. Table 6.2 shows all of the periods of hindcast and measured wave conditions with corresponding percentage ratios of the record-averaged littoral drift average annual RMS values to the corresponding 40-year value.

Table 6.1 Root Mean Square of the Long Shore Discharge Values for the Q_{long} From 1960 To 2000 (down-coast = +, up-coast = -)

Year	Q_{long} Root Mean Square	Root Mean Square For Down-Coast Transport	Root Mean Square For Up-Coast Transport	Down-Coast and Up-Coast Difference
1960	1864	594	1767	-1173
1961	1640	217	1626	-1409
1962	764	317	695	-378
1963	489	180	455	-275
1964	1613	189	1602	-1413
1965	701	437	548	-111
1966	496	216	446	-230
1967	992	801	587	214
1968	529	259	461	-202
1969	523	317	416	-99
1970	562	278	489	-212
1971	803	408	692	-285
1972	596	431	411	20
1973	617	419	453	-34
1974	393	217	327	-110
1975	733	379	628	-249
1976	824	543	620	-78
1977	794	348	713	-364
1978	631	175	607	-432
1979	474	215	422	-207
1980	813	702	409	294
1981	565	300	479	-179
1982	518	228	465	-237
1983	633	231	589	-358
1984	1024	349	962	-613
1985	780	349	698	-349
1986	520	200	480	-279
1987	482	166	452	-287
1988	624	326	532	-206
1989	820	209	793	-584
1990	951	266	913	-647
1991	828	315	765	-450
1992	592	208	555	-347
1993	735	259	688	-430
1994	768	552	533	19
1995	816	545	608	-63
1996	1042	676	793	-117
1997	706	567	420	147
1998	868	725	477	248
1999	577	363	448	-86
2000	631	372	511	-139
40-yr Average=	**764**	**362**	**647**	**-285**

Table 6.2 Average Root Mean Square for Whole Q_{long} Record

Data Set Description	Period of Data From:	To:	Root Mean Square Average	Ratio of Period's RMS versus the total RMS
'New' WIS	1/1/1976	12/31/1995	710	0.93
'Old' WIS	1/1/1960	12/31/1975	832	1.09
OCTI	1/1/1970	1/1/1979	643	0.84
LATX	1/1/1992	1/1/1994	699	0.91
NOAA Buoy #42020	5/1/1990	12/31/1995	782	1.02

7.0 APPLICATION OF MODELING RESULTS

The average-annual littoral drift RMS values listed in Table 7.1 can be used to estimate the relationship between average channel sedimentation rate that was computed with the GENESIS model using the 20 years of ‘updated’ WIS data and the long-term average conditions. Sedimentation during periods of more severe conditions can also be estimated. Again the ratios of the average annual littoral drift RMS values is used to normalize the results. This ratio for the full 40-year period divided by the period corresponding to the 20-year ‘updated’ WIS data interval is 1.08 cy/yr. This indicates that the average annual channel sedimentation rate of 38,000 cy/yr computed during the ‘updated’ WIS data time interval can be adjusted to a rate of 41,000 cy/yr for the 40-year average conditions.

Table 7.1 lists corresponding corrected annual sedimentation rates in descending order for one, two, three and four year intervals. The multi-year intervals were selected according to their respected summed channel sedimentation rates. For all entries the year(s) are shown above and the corresponding sedimentation rates are shown below in parentheses. Recall the effects of cyclonic storms and aeolian sand transport have been removed from our analyses.

Consider the annual data shown on Table 7.1. The top entry is the maximum annual rate encountered in the 40-year sequence. It is expected to be equaled or exceeded once in 40 years. The second entry is expected to be equaled or exceeded twice in 40 years (i.e. 20-year recurrence interval) and so on down the table. Similar considerations apply to the multi-year data. The worst two years are given at the top of the third column. This is the maximum two-year sedimentation rate in the 40-year record and thus the two-year rate with a 40 year recurrence interval. The other recurrence interval values are given on the table. Depending on whether annual or multi-year dredging intervals are considered the values given in Table 7.1 can be used to estimate the severe condition contingency value. For example, if dredging each two years is contemplated, and it is decided that this should be planned to allow for the amount of sedimentation that could occur in the channel with a risk of the 10-year reoccurrence interval, then the value is 120,000 cy (e.g. 2-year column, 10-year recurrence interval table entry). This is the expected 2-year total excluding any contribution from wind-blown sand and not considering cyclonic storms.

Table 7.1 Estimated Channel Sedimentation Rates by Dredging and Recurrence Intervals

(without aeolian transport or effects of cyclonic storms)

Re-occurrence Interval (years)	One Year		Two Year Sequence		Four Year Sequence	
	Year	Channel Sedimentation ($x10^3$cy)	Year Pair	Channel Sedimentation ($x10^3$cy)	Year Sequence	Channel Sedimentation ($x10^3$cy)
40	1960	124	1960-61	163	1960-64	398
20	1961	110	1964-65	158	1995-98	230
10	1996	72	1984-85	120	1988-91	217

8.0 JETTY FUNCTIONAL CHARACTERISTICS

8.1 WAVE – CURRENT INTERACTIONS

A wave-current interaction analysis has been conducted to determine the impact of the jetties on wave steepening and potential breaking at the entrance to the jetties. The analysis is divided into two parts: (1) the effects on wave steepness and (2) the effects on wave breaking due to both depth-limited breaking and/or steepness-induced breaking. The primary affect of the jetties on wave steepness and wave breaking is to to cause the ebb and flood tidal flows to interact with the waves at the 11 ft mean water depth. The wave-current interaction causes the waves to increase in height and steepness (ebb tide) or to decrease in height and steepness (flood tide).

In order to provide a representative sample of waves and tidal flows, the wave records in the WIS 1087 hindcast analyzed (over 49,000 individual waves). Large waves associated with named cyclonic storms were removed from the wave data set prior to the analysis. A tidal velocity was assigned to each wave (both ebb and flood) by recreating the diurnal and spring/neap cycle of channel velocities. A cycle similar to that predicted for the jetty entrance in a previously completed Corpus Christi Bay modeling study (Brown and Militelo, 1997) was used. The application of this cycle should provide stable statistical values, since over 49,000 records were used (20 years of hourly data). For each record, the waves were transformed from deep water to shallow water (11ft) using linear wave theory.

The effects of the jetties on wave steepness were estimated by adjusting the wave height and length due to the presence of the tidal flow using the shallow-water approximations to the wave-action equation. The wave steepness, defined as the wave height over the wave period, was then calculated for the transformed waves, with and without, the effects of the currents.

Table 8.1 Effects of the Tidal Flow on Wave Steepness

Hs (ft)	Percent Occurrence (%)	Maximum Steepness	
		With Jetties	Without Jetties
0 to 1	4.30	0.023	0.017
1 to 2	46.83	0.047	0.034
2 to 3	35.98	0.062	0.050
3 to 4	10.22	0.074	0.059

4 to 5	2.01	0.093	0.070
5 to 6	0.51	0.113	0.083
6 to 7	0.16	0.125	0.095

The maximum steepness is increased about 35%, and is about the same for all wave heights. The maximum wave steepness that occurs is 0.125 corresponding to waves in the 6 to 7 ft range.

The effects of the jetties on wave breaking were also accessed, for both steepness induced breaking and depth limited breaking. For steepness induced breaking the steepness for each wave record were compared to the steepness criteria:

$$\left(\frac{H}{L}\right)_c = 0.142\ \boldsymbol{tanh(}2pd / L\boldsymbol{)}\ .$$

Here H is the wave height, L is the wave length and d is the water depth. If the actual H/L ratio exceeds the critical value $(H/L)_c$, the waves are assumed to break.

The potential for depth-limited breaking was determined by comparing the ratio of the predicted wave height and water depth to a breaking criteria. The breaking criteria is a function of the deep water wave parameters and beach slope (Gravens and Kraus, 1991). For the analysis of depth-limited wave breaking, the wave records, which represent the significant wave height, Hs, were adjusted to reflect the wave heights associated with H_1 (the average of the highest 1 percent of waves) before comparing to the breaking criteria. The results are tabulated in Table 8.2.

Table 8.2 Estimated Wave Breaking Statistics

Percentage of Time Waves Break		
	Depth Limited	**Steepness**
Without Jetties	0.051	0.28
With Jetties	0.085	0.48

The results indicate that depth limited wave breaking without the jetties is only expected to occur about 5 hours a year (~0.05% of the time). The presence of jetties increases this time an additional .034 percent (about 3 hours). Breaking due to wave steepness is calculated to occur about 24 hours a year in the absence of the jetties, and

42 hours per year with the jetties. The average wave height associated with the depth limited breaking is on the order of 6 ft, whereas the steepness including breaking occurs for a wider range of wave heights. The average height of waves that were predicted to break due to steepness effects is 2.5 ft.

The length of the jetties and design depth appears to be sufficient and do not promote significant increase in navigation hazards. Wave breaking will occur during some periods, but is brief. During periods of increased depth limited breaking, the incoming waves heights are equal to or greater than 6 ft.

8.2 INLET STABILITY

Inlet stability analyses have been conducted for the proposed re-opened Packery Channel to evaluate its potential stability characteristics. Various stability analysis have been conducted in the literature for both the proposed Packery Channel and for Mansfield Pass. Kraus and Heilman (1996) have conducted four analyses, the Tidal Prism and the Width-to-Depth Methods described in Jarrett (1976), the Bruun Ratio Method (Bruun 1991) and the Scour Velocity Method (Bruun 1990 and 1991). Hayes et al. (1997) conducted an Escoffier analysis for the proposed Packery Channel. Kieslich (1997) conducted the Tidal Prism, Bruun Ratio and Escoffier Analysis for Mansfield Pass. We have reviewed each of the analysis and compared the results for four of the methods (Tidal Prism, Bruun Ratio, Scour Velocity and Escoffier Methods). Mansfield Pass is generally similar to the proposed re-opened Packery Channel in many regards and a comparison of the results from the stability analysis combined with known characteristics at Mansfield Pass provide a means for accessing conditions at the proposed Packery Channel.

For Packery Channel, the tidal prism analysis was based on a tidal prism of $1.81x10^8$ sq. ft. prism, equivalent to the basin associated with the Upper Laguna Madre. The associated critical channel cross-sectional area is $1.97x10^3$ sq. ft. The proposed channel is assumed to have a cross-sectional area of $1.9x10^3$ sq. ft, indicating a stable configuration. For the Bruun Ratio, the same tidal prism was assumed, with a gross longshore transport of 200,000 cy/yr (5,400,000 ft^3), yielding a tidal prism to transport ratio of 12. This value is considered to be in the ‘unstable range’. When the higher gross transport rate derived in this study, 365,000 cy/yr is used, the ratio becomes even smaller, yielding a less favorable stability assessment. In the scour velocity method, velocities on the order of 3 ft/s are required to assure a stable inlet or channel. Results from a hydrodynamic study of the proposed Packery Channel (Brown and Militelo, 1997) indicate that channel velocities will reach or exceed 3 ft/s during spring tides, and thus the proposed channel may be stable. In the Escoffier analysis for Packery Channel, the critical velocity was assumed to be 3.0 feet, and the results, for reasonable ranges for the Gulf tidal amplitude, 1 to 2 ft, indicate that the channel will be unstable.

For Mansfield Pass, the tidal prism is estimated to be $1.52x10^8$ cu. ft., based on velocity measurements taken over tidal cycle, and yields a critical cross-sectional area of 3,730 sq. ft. (if the exact formulation used for Packery Channel is used, the critical area becomes: 4070 sq. ft). The actual cross-sectional area, based on measurements after dredging the channel, is 5100 sq. ft. Based on these areas, the Tidal Prism Method indicates that the channel is unstable. Peak velocity measurements in the channel typical ranged from 0.8 to 2.67 ft/s (Kieslich, 1977) which are below the 3.0 ft/s criteria in the Scour Velocity Method, indicating an unstable channel. In the application of the Bruun Ratio to Mansfield Pass, a gross transport of 1,091,250 cy/yr (Kieslich, 1997) yields a ratio of 5, indicating that there will be minor instability. Using the gross transport derived in this study (503,000 cy/yr), the ratio is 10, and also indicates an unstable rating. The Escoffier analysis for Mansfiled Pass, using 3.2 ft/s as the critical velocity indicated that the channel is unstable. The results of applying all four methods to the proposed re-opened Packery Channel are given in Table 8.3.

Table 8.3 Results of Stability Analysis

Method	Packery Channel	Mansfield Pass
Tidal Prism	Stable	Unstable
Scour Velocity	Marginally Stable	Unstable
Bruun's Ratio	Unstable	Unstable
Escoffier	Unstable	Unstable

There is no clear trend in the results of the four methods. Overall, Packery Channel could be considered to rate better than Mansfield Pass, but only for two for the four methods. It is known from dredging records at Mansfield Pass that the channel is unstable, and significant channel shoaling does occur. Therefore, as a conservative approach, we assume that the flows in the Packery Channel will not be sufficient to maintain the channel, and shoaling will occur.

9.0 SAND BYPASSING OPERATIONS

9.1 IMPACT OF SPUR JETTIES ON SAND BYPASSING OPERATIONS

An effective sand management plan will be critical to ensure the long-term success of the proposed Packery Channel project. Previous studies have identified sand bypassing as a key component to such a management plan. A separate report (Dredge Material and Sand Management Plan, 2001) has been prepared and submitted by URS. Some relevant issues are summarized briefly in the following sections.

9.1.1 General Discussion on Sand Bypassing Operations

Sand bypassing involves moving the sand that accumulates either in the area updrift of the jetty or in the channel. Bypassing is typically accomplished through one of the following dredging operations:

- A bypassing plant located updrift of the jetty;

This type of bypassing operation is intended to bypass the sediment before it has a chance to enter the channel. Consequently, this approach limits shoaling and the associated hazard to navigation within the channel.

Bypassing plants consist of a dredging system that operates in the area immediately updrift of the jetty. The system may be composed of a fixed plant that is permanently constructed at the site or a mobile plant that can be moved to and from the site. The dredging is accomplished using eductor pumps, suction pumps or submersible pumps. A fixed plant configuration typically consists of a structure constructed on or near the updrift jetty with a boom that can deploy a pump or suction pipe to the area where the updrift fillet accumulates. The reach of the boom is limited to the area directly in front of the plant. Mobile plants are similar except that the pump or suction pipe is deployed from a crane, which can move up and down the updrift beach within certain limits.

Some bypassing plants have been constructed to include a pier extending out into the surf zone of the updrift beach, allowing access to more of the sand. Crawling systems have also been developed which can “walk” the dredge pipe out into the surf zone and beyond.

- A hydraulic cutterhead dredge working in the channel; and

Hydraulic cutterhead dredges are a standard piece of dredging equipment, and are very efficient at dredging sands under the appropriate conditions. A typical application of hydraulic dredges in a bypassing operation involves using the dredge to remove sand that has accumulated in the entrance channel and placing it on the downdrift beach. A major disadvantage of hydraulic dredges for sand bypassing is their limited ability to operate in waves. Hydraulic dredges that can operate in open water subject to wave action are typically large and require an operating depth on the order of 10 to 15 feet.

- A hopper dredge working in the channel or on the shoals near the entrance to the jetties.

Hopper dredges can be used to dredge material from the channel and offshore areas adjacent to the entrance channel. They have the advantage of being able to operate effectively even in wave environments. However, unloading the material from the hopper can be problematic if the material is to be placed on downdrift beaches. In such cases, the dredge must use a pumpout system which can be slow and, ultimately, very expensive. In addition, hopper dredges are generally large vessels requiring operating depths of 10 ft or more. Because of their size, they can be difficult to operate in confined channels.

9.1.2 Application of Spurs to Packery Channel

The jetty configuration for Packery Channel recommended in the Blucher Institute study (Kraus and Heilman, 1997) includes spur jetties on the outward side of the main jetties, near their offshore tips. Spur jetties are a relatively recent innovation in coastal engineering, and the real-world experience on their performance is limited. Studies of this configuration indicate that the spur jetties would deflect the currents that form along the outer edges of the jetties and create an eddy in the area between the spurs, the jetty and the shoreline. The Blucher Institute study states that "… 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." As indicated in this statement, how the spur jetties will impact the sediment deposition is uncertain. However, it is generally speculated that the sediments will form a broad shoal offshore of the spur jetties. The scenario envisioned in the Blucher Institute study is that the sand will "stockpile" in the area between the jetties, spurs, and shoreline. Bypassing would be accomplished by using a hydraulic cutterhead dredge to remove stockpiled sediments and pumping them to the downdrift beaches.

9.1.3 Feasibility of Bypassing "Stockpiled" Sands

As part of the overall evaluation of whether or not to include the spurs in the final design, the feasibility of bypassing the sand stockpiled by the spur jetties must be considered. The primary concern is how the sand that accumulates in the updrift area would be bypassed.

The Blucher Institute study recommends the use of a hydraulic dredge to perform the bypassing in association with the channel dredging. As noted above, hydraulic dredges are not well suited for operating in open waters where they will be exposed to waves. Wave action can cause severe damage to the spuds and ladders. In addition, waves make the handling of the pipelines required for hydraulic dredges very difficult and dangerous. It is possible that under very calm conditions a hydraulic dredge could safely perform the dredging; however, the standby time associated with waiting for a suitable window would likely be unacceptable. While larger hydraulic dredges are available that can operate in wave conditions, the water depths in the areas where the dredging needs to occur would not be sufficient for the dredge to operate. Due to these operational constraints, use of a hydraulic dredge to bypass the sand in the offshore shoals associated with the spurs is considered impractical.

Hopper dredges would also be poorly suited to dredge the sand “stockpiled” by the spur dikes. It is unlikely that the hopper dredges would be able to operate in the water depth associated with these deposited sands. Furthermore, the proximity of the spur dikes to the shoals would likely make it difficult or impossible to maneuver a hopper dredge into the area where the shoals have formed. The need to pump out the material for beach placement would add a considerable amount to the dredge cost.

Since the spur dikes will likely result in the creation of large shoals positioned several hundred feet offshore, the majority of the sand would be beyond the reach of a fixed or even a mobile sand bypass plant. Furthermore, the sand will likely be spread over a relatively large area, which is not conducive to sand bypassing operations.

9.1.4 Summary of Spur Jetty Impacts

If the spur jetties perform as anticipated, the sand will form a shoal in the area between the spur jetty, the jetty and the shoreline. As described above, dredging of this shoal could prove to be very difficult. Therefore, the spur jetties may actually have a negative impact on bypassing operations at the site. Consequently, the spur jetties are not recommended from the perspective of sand bypassing.

The results of the analysis indicate that Packery Channel is likely to be unstable and therefore the tidal flows will not be able to keep the jetty entrance from shoaling. The analysis also indicates that Packery Channel will have similar stability characteristics to Mansfield Pass. Thus, the channel will need regular maintenance dredging.

Sand will need to be regularly dredged from the channel and placed on the down-drift beach. Sand should also be bypassed from the up-drift fillet to the down-drift beach. A jet eductor pump suspended from a long-boomed crane has been used successfully at the Indian River Inlet, Delaware (Dean, 2000 personal communication). Because of the possibility of variable net longshore drift directions at the proposed Packery Channel, this bypass system could be used.

10.0 LIMITATIONS

Analyzing coastal processes and the affects of coastal structures with numerical models is a relatively new and still developing practice. Various forms of data and parameter summaries are needed which make the modeling results take on a generalized character. Also, future realizations of the wave climate, beach shape other factors can only assume to be similar to past conditions. Given the importance of short-term antecedent conditions on the specific patterns of coastal changes, these assumptions of likely future conditions is a significant limitation in the analyses. The work summarized in this report has been performed according to the normal practices of the professions. The data sources, assumptions and analytical methods are stated in the report. The reader must make independent assessments of how the results are to be used and where allowances are needed to address uncertainties in the results.

11.0 REFERENCES

Brown, Cheryl A.. and Adele Militello. 1997. Packery Channel Feasibility Study: Bay Circulation and Water Level. Prepared for Naismith Engineering. April 31, 1997.

Bullard, F.M., 1942. Source of Beach and River Sands on Gulf coast of Texas. Bulletin of the Geological Society of America, 53: p 1021-1044

Carothers, H.P. and H.C. Innis. 1960. Design of Inlets for Texas Coastal Fisheries. Journal of the Waterways and Harbors Division, Proceedings of the ASCE, 127(IV): p 231-259

CERC Shore Protection Manual, Volume I, 4Th ed. 1984. USACOE

Dames & Moore. 2000. Packery Channel, Texas, North Padre Island Storm Damage Reduction and Environmental Restoration Project. Technical review and project report. Unpublished consulting report

Dean, R.G. Personal communication, September 21, 2001

Hayes, M.O., Van de Kreeke, J. and R.G. Dean. 1997. Packery Channel Opening: Peer Review Panel Assessment. Texas General land Office. Austin, Texas.

Gravens, Mark B. and Nicholas C. Kraus., Genesis: Generalized Model For Simulating Shoreline Change. Report 2, USCOE TR CERC-89-19. September, 1991.

Hayes, M.O., van de Kreeke, J., and R.G. Dean. 1997. Nueces County Recreational Capital Improvement Program, Packery Channel Opening: Peer Review Panel Assessment. Consulting Report.

Kieslich, James M. 1997. A Case History of Port Mansfield Channel, Texas. USCOE GITI Report 12. May, 1977.

Komar, P.D. 1976. Beach Processes and Sedimentation. Prentice Hall, Upper Saddle River, .J., 544 pps.

Kraus, Nicholas, C. and Daniel J. Heilman. 1997. Packery Channel Feasibility Study: Inlet Functional Design and Sand Management. Prepared for Naismith Engineering. March 31, 1997.

Paine, J.G. and R.A. Morton. 1989. shoreline and Vegetation-Line Movement, Texas Gulf Coast, 1974 to 1982. Bureau of Economic Geology Geological Circular 89-1, University of Texas at Austin. Austin, Texas. 60 pps

Price, W.A. 1933. Role of Diastrophism in Topography of Corpus Christi Area, South Texas. American Association of petroleum Geologists Bulletin, 17(8): p 907-962

URS, 2001, Dredge Material and Sand Management Plan, unpub. Consulting Report.

U.S. Army Corps of Engineers. 1984. Shore Protection Manual, Vols. 1 and 2. Waterways Experiment Station, Coastal Engineering Research Center

Van Andel, T.H. and D.M. Poole. 1960 Sources of Recent Sediments in the Northern Gulf of Mexico. Journal of Sedimentary Petrology, 30(1): p 91-122

Watson, R.L. 1971. Origin of Shell Beaches, Padre Island, Texas. Journal of Sedimentary Petrology, 41(4): p 1105-1111

Watson, R.L. and E. W. Behrens. 1976. Hydraulics and Dynamics of New Corpus Christie Pass, Texas: A Case History, 1873-75. GITI Report 9. Prepared for U. S. Army Coastal Engineering Research Center, Vicksburg, Mississippi, p 175

Weise, B.R. and W.A. White. 1980. Padre Island National Seashore, A Guide to the Geology, Natural Environments and History of a Texas Barrier Island, Guidebook 17. Bur. Eco. Geol Austin, Texas. 94 pps.