Corpus Christi Ship Channel Mustang and Harbor Islands Shoreline Stabilization Project

Texas General Land Office

Pacific International Engineering PLLC

Corpus Christi Ship Channel
Mustang and Harbor Islands
Shoreline Stabilization Project

Texas General Land Office

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1. Introduction

Pacific International Engineering, PLLC (PI Engineering), under contract with the Texas General Land Office (GLO), has conducted a study "Corpus Christi Ship Channel Shoreline Stabilization Project." The objective of the project is to reduce and –if possible–prevent the loss of property and public infrastructure, enhance the usability and accessibility of public areas for recreational purposes, preserve and enhance existing coastal wetlands and habitats, and reduce shoaling within the navigational channel. The areas of concern include Harbor Island and Mustang Island along the Corpus Christi Ship Channel. Specific sites of concern within project area are as follows:

Harbor Island Shoreline

- Levee containing the dredged material placement area No. 4. Erosion at the levee threatens harbor facilities and increases maintenance dredging.

Mustang Island Shoreline

- Watermain project. Protection is required for a 20-inch water supply line and adjacent shoreline;
- Public fishing pier and park shoreline approximately 1,000 ft toward the west. Historically, broken concrete rubble has been randomly along the current shoreline, but this shoreline protection measure has not been successful;
- Access road to the park site. Erosion threatens the access road where the road meets the fishing pier. Damage to the road may occur if erosion continues adjacent to the east of the pier; and,
- Public access corridor along the shoreline, from the west of the Public Fishing Pier Project, toward Piper Channel. Erosion along the shoreline is causing extensive land loss and threatens adjacent wetlands.

This study includes historical data collection and review, new field data collection, numerical modeling and analysis, development of alternatives, engineering calculations, conceptual drawings and preliminary cost estimates. The study commenced on July 5, 2000. Since then, the following technical and coordination efforts have been completed:

- July 10, 2000 - Meeting with the City of Port Aransas and the GLO. The study and coordination plans were discussed and agreed upon. It was decided that the project schedule should be re-evaluated to expedite completion of the alternative analysis and selection of the preferred alternative(s) for the Watermain Project.

- July 10, 2000 – PI Engineering prepared and forwarded Meeting Minutes to the GLO (Appendix A).
- July 11, 19, and 20, 2000. - PI Engineering conducted site visits which included photographing the existing shoreline to develop the shoreline inventory.
- July 21, 2000 – PI Engineering issued and submitted a draft Project Review Report to the GLO. The Project Review Report summarized the project history and findings from previous studies. In addition, the draft Project Review Report provided a review and analysis of the permit application and supporting documents prepared by Shiner and Moseley Associates, Inc. for the City of Port Aransas.
- August 8, 2000– PI Engineering issued a final Project Review Report to the GLO (Appendix B).
- August, 2000 – PI Engineering conducted bathymetry and topographic surveys for specific areas of concern, at Harbor Island and Mustang Island.
- August, 2000 – PI Engineering deployed a wave gage for the collection of wave data at the shoreline adjacent to the Watermain Project.
- August 4, 2000 PI Engineering conducted field experiments by collecting data from deep-draft vessels including vessel speed and position in the channel, and vessel generated waves.
- August 11, 2000 – PI Engineering presented a range of potential alternatives for the Watermain Project at a meeting with the GLO in Austin, Texas.
- August 11, 2000 – PI Engineering prepared and presented a Shoreline Inventory of the Corpus Christi Ship Channel (CCSC). The inventory consists of a basemap -aerial photograph of 1995 and ground-level photographs of the shoreline, which were collected during site visits. The City of Port Aransas shoreline inventory was presented in hard copy and in digital format (Appendix C).
- August 24, 2000 – PI Engineering met with the City of Port Aransas and the GLO to discuss the Watermain Project Alternatives.

This report presents the results of the study for Tasks 2 through Task 4 of the Scope of Services including investigating the causes of erosion, developing a range of alternatives to stabilize existing beach erosion, alternative analysis and recommendations for the preferred alternatives. Previous studies and history of the project in general are presented in the Project Review Report (Appendix B). The specific results of previous studies that are relevant and used in this report are presented in relevant sections.

Following a meeting on July 10, 2000 with the GLO, City of Port Aransas and the City of Corpus Christi, a decision was reached to make the Watermain Project a priority. This report presents alternatives to stabilize the pipeline and shoreline along the Watermain Project area only. Shoreline erosion protection alternatives for the areas of: Harbor Island, Access Road, Public Park at Fishing Pier, and Public Access along Mustang Island will be presented in subsequent amendments to this report.

2. Causes of Erosion

Prior to developing the Corpus Christi Shoreline Stabilization alternatives, an effort has been made to identify and quantify the coastal processes and mechanisms that cause shoreline erosion. Two major hydrodynamic factors presumably responsible for the erosion have been investigated: tidal currents and deep-draft vessel impacts. The importance of these factors, and forces generated by these factors, were indicated in the previous study (see Project Review Report, Appendix B), discussions with the Members of the Corpus Christi and Port Aransas Ship Channel Pilot Association, and local residents.

2.1 Deep-Draft Vessel Impacts

Two techniques were used to investigate hydrodynamic forces on the seabed generated by deep-draft vessels travelling through the CCSC and potential impacts from these forces on shoreline erosion: field data collection and numerical modeling.

2.1.1 Field Data

Field data were collected during the period of August 3rd through August 10th using a SeaBird non-directional pressure sensing wave gage. The pressure gage was deployed along the edge of the ship channel (Figure 2-1) at an elevation of –6 ft (MSL) and practically continuous recorded pressure oscillations at a frequency of 4.0 Hz (once every ¼ second). Data were processed using standard procedures and analyzed to identify wave height and period for all wave events recorded by the gages.

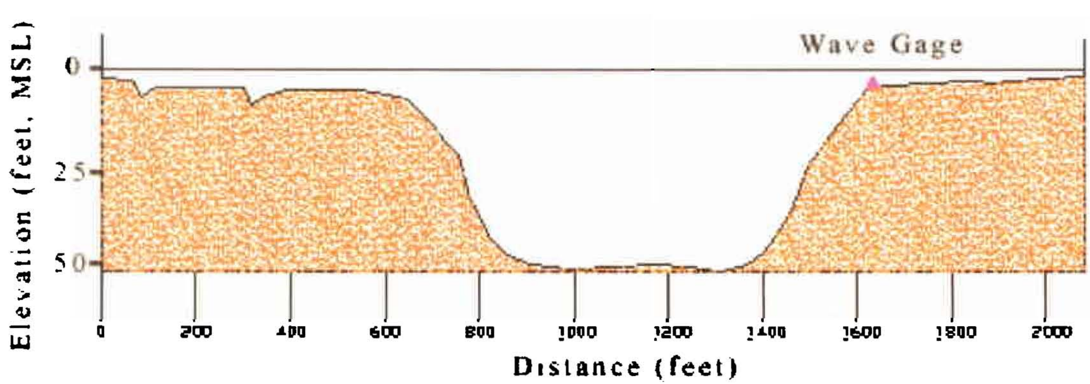

Figure 2-1 Wave Gage Location in the CCSC Cross-Section

Analysis shows that the pressure gage wave record consisted of wind-generated waves, deep-draft vessel wakes and pressure field effects, and wakes generated by small craft. Examples of wind-generated waves, deep-draft vessel wakes, and deep-draft pressure field effects are shown in Figure 2-2. Wakes generated by a small-craft vessel are shown in Figure 2-3. The maximum wave height was measured on August 6, 2000 at 12:24, and was approximately 2.9 ft.

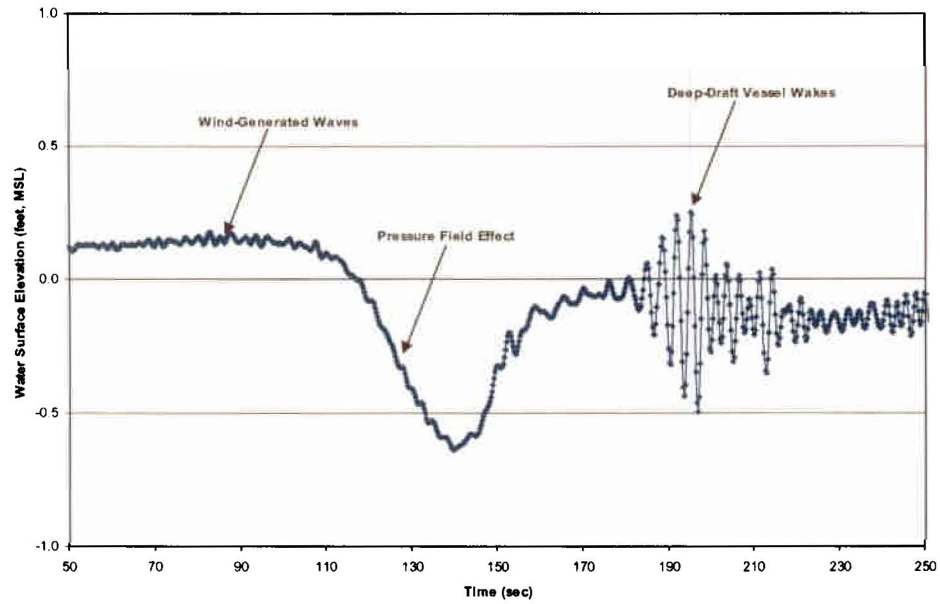

Figure 2-2 Example Measurement of Various Types of Waves in the CCSC

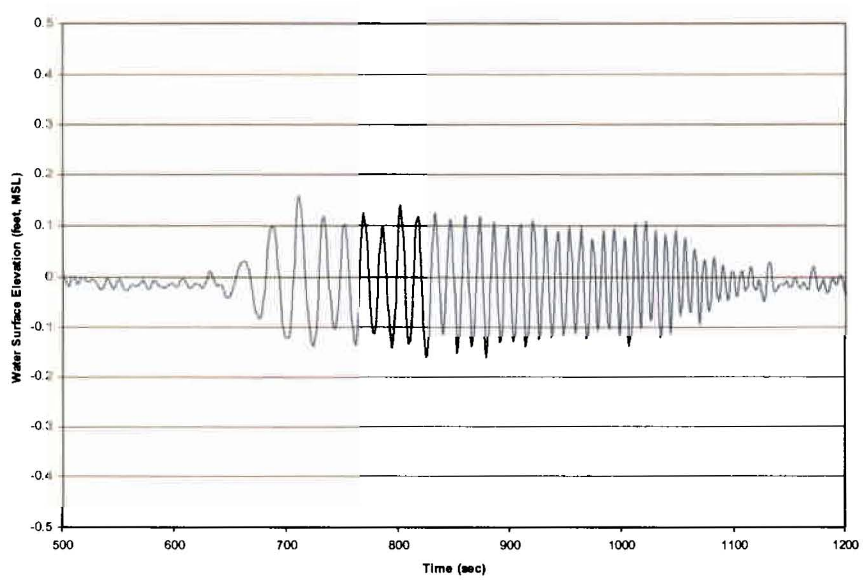

Figure 2-3 Example Measurement of Small-Craft Vessel Wake in the CCSC

The largest wave heights and periods recorded by the gages are mostly generated by deep-draft vessels and pleasure boats. Wind-generated waves have minimal contribution to the wave energy flux delivered to the shoreline along the CCSC at the City of Port Aransas.

The recorded data demonstrate the importance of pressure field effects from deep-draft vessels in terms of hydrodynamic forces delivered to the shoreline. Pressure field effects are generally created by deep-draft vessels moving through restricted waterways. While moving through a restricted waterway, a pressure differential forms along the body of vessel; low-pressure areas develop along the sides, and high-pressure areas develop in front of the bow and behind the stern. Pressure field effects move along with the passing vessels and are projected along the waterway banks and the shoreline in the form of long-period waves (wave period on the order of several minutes and wave height commonly exceeding two-three ft). The pressure field wave hydrodynamics may translate into significant loads and forces along the waterway. The following is a quotation from the U.S. Section of the International Navigation Association (PIANC):

> "The pressure pulse may, however, be of inconvenience for moored ships as the ship may be set in motion. In case of relatively high maneuvering speeds of the passing vessel, the problem can be very serious."

Analysis of the field wave data demonstrates that the Corpus Christi Ship Channel, along the City of Port Aransas, is a location where strong pressure field effects are created during the passage of deep-draft vessels. The pressure field effect is known locally as a "drawdown". It was described (also recorded on videotape) as a two-phase water movement; first, the water is drawn toward the channel, exposing the bottom slope down to elevations -2 to -4 ft (MSL). Second, the water rushes back up the slope (return flow) towards the bluff[1] location (most of the time in the form of a high-speed bore). It has been observed by local residents that this type of water movement is probably a major factor contributing to the shoreline erosion along the Corpus Christi Ship Channel at the City of Port Aransas.

To maintain position with the CCSC waterway boundaries, pilots navigating deep-draft vessels must maintain sufficient speed, commonly exceeding 6-8 knots. As a result, pressure field effects are created in the CCSC under certain combinations of environmental

[1] Most of the unprotected shorelines along the CCSC have bluff, which are the result of shoreline erosion and beach retreat. The bluff is usually located at the upper beach and is a vertical sandy wall of two to six feet in height.

conditions (tidal elevation, current direction, vessel position and vessel speed). The field data show that pressure field effects at the CCSC can be represented by long-period waves (periods 80 to 120 seconds) with measured wave heights between approximately 0.5 and 4.0 ft.

PI Engineering experience shows (Shepsis et al, 1998) that pressure field wave parameters (wave height and period) depend on a number of factors such as vessel draft, vessel speed, water depth, current velocities in the channel, vessel dimensions, etc. Pressure field wave heights may increase dramatically with relatively small changes in some of the aforementioned parameters. From the perspective of developing the shoreline erosion protection alternatives, it is important to determine the extreme conditions (largest waves) that can be generated by pressure field events.

Analysis of the CCSC vessel traffic data shows that pressure field waves, recorded during field measurements, most likely represent a relatively small range of potential waves that may occur in the channel. The limited time of observation did not allow determination of the most critical conditions that may exist along the shoreline. Therefore, a numerical modeling effort was completed to simulate design pressure field conditions that may occur in the channel. The data collected during the field data collection program were used to calibrate and validate the model. The field data collection program was augmented with an experiment designed to provide reliable information on deep-draft vessels passing through the channel. The field experiment consisted of tracking deep-draft vessels as they passed through the CCSC in the vicinity of the wave gage station. This experiment provided information on vessel parameters (length, beam, draft), vessel speed, and vessel location relative to the channel centerline. A total of seven deep-draft vessel-tracking experiments were conducted. The results are presented in Table 2-2.

Table 2-2 Vessel Tracking Experiment Data

Ship Name	Length (ft)	Beam (ft)	Draft (ft)	Speed (kts)	Direction	Tracking Position
Monarch	618.1	101.6	26.1	9.9	Inbound	100' astern, 10' stbd
CEM Princess (Light Chemical)	402.5	59.0	23.3	9.0	Inbound	50-75' astern, 5-10' stbd
Henning Maersk I (Light Chemical)	525.0	84.0	24.0	11.9	Inbound	100' astern, 10' stbd
Texas	500.0	78.0	26.3	7.9	Inbound	50-75' astern, 5-10' stbd
Molda (Tanker)	761.3	137.7	38.5	8.3	Inbound	100' astern, 10' stbd
Maritrans	525.0	84.0	18.0	11.4	Inbound	100' astern
Teseo (Tanker)	800.0	150.0	28.0	8.7	Outbound	100' astern, 10' stbd

The data from Table 2-2 are used in combination with the measured pressure field parameters to validate the numerical model. Three of the vessels tracked in the field experiment had produced pressure field waves that were recorded by the wave pressure gage and are used in Section 2.1.2.2.

2.1.2 Numerical Modeling

2.1.2.1 Numerical Modeling Description

The Vessel-Generated Pressure Field (VGPF) model was selected to simulate the pressure field waves in the Corpus Christi Ship Channel. The VGPF model is a 3-D hydrodynamic numerical model that simulates pressure field distributions and calculates water surface elevation fluctuations in the waterways during deep-draft vessel passage.

The VGPF model was developed by PI Engineering and the National Institute of Hydrodynamics, Academy of Science, Ukraine. The VGPF model uses slender body theory (Shepsis et all, 1998) and the method of matched asymptotic expansion to calculate a 2-dimensional pressure distribution in the waterway. The model assumes that the passing vessel is an ellipsoid with a certain length, beam and draft. Figure 2-4 shows an example of spatial pressure field distribution calculated for a deep-draft vessel moving in a restricted waterway near the channel bank (the passing vessel is displaced from the channel centerline).

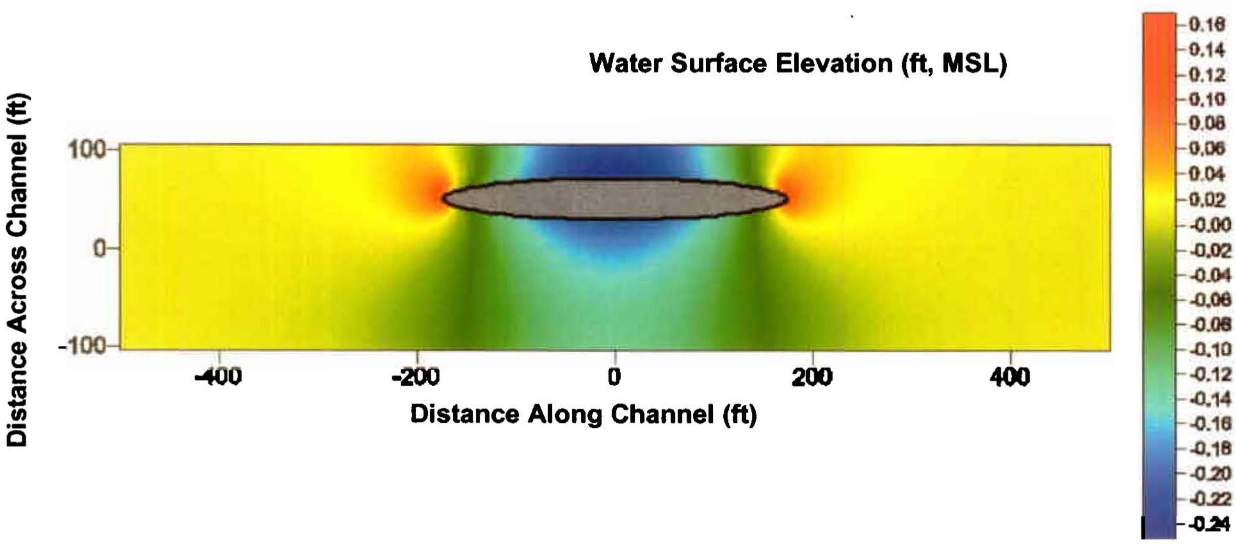

Figure 2-4 Example Water Surface Elevation Distribution in Restricted Waterway Due to Pressure Field

The VGPF model was validated using wave data measured during the field data collection program in the CCSC (see Section 2.1).

2.1.2.2 Model Validation

To validate the models, numerical modeling was conducted for the trial pressure field events. For each pressure field event, a water surface elevation time series was computed and compared to the measured data. Figure 2-5 combines two graphs of: the measured long-period wave generated by the Henning Maersk on August 4, 2000 at 18:05 and the long-period wave calculated by the VGPF model. There is good agreement between the predicted and measured long-period wave heights for this event. The measured and predicted maximum drawdowns for three available measured waves are shown in Figure 2-6. The scatter in the data could be due to some error in vessel shape, position, speed, and/or other vessel traffic in the waterway.

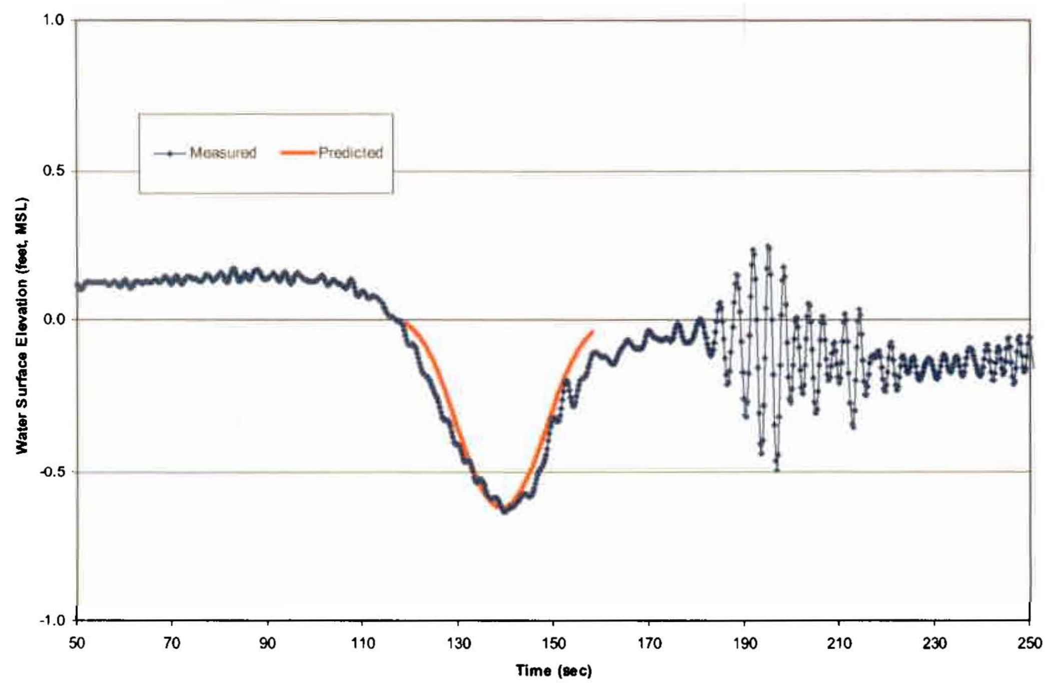

Figure 2-5 Measured and Calculated Pressure Field Wave for Vessel "Henning Maersk" on August 4, 2000 at 18:05.

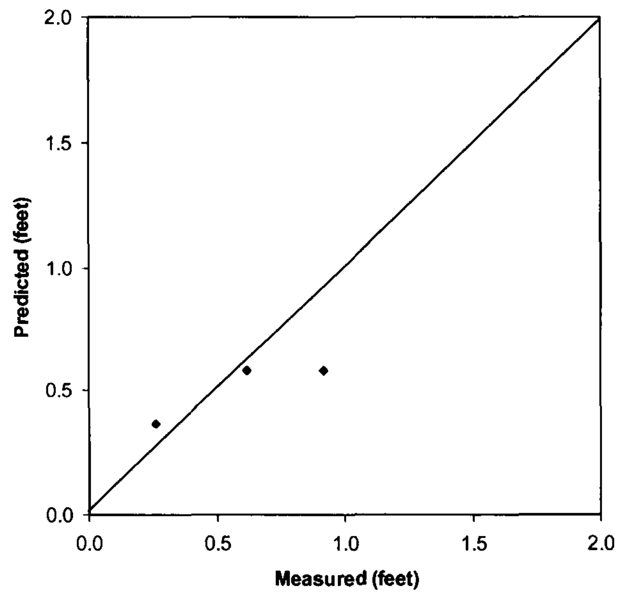

Figure 2-6 Measured and Calculated Pressure Field Wave Heights

Based on the comparison between field data and computed results, the VGPF model has been considered verified for the CCSC conditions. Although data is limited (only 3 are available) for the comparison, previous efforts on model calibrations in similar projects was considered. It is suggested that in general, the model reproduces long-period pressure field waves reasonably well.

The design conditions for the numerical modeling were selected based on the following considerations. The design pressure field conditions are considered to be long-period waves, generated by the largest recorded vessel moving through the CCSC at approximately 10 knots. The selected design vessel is the "Algarrobo", with length 923 ft, beam 176 ft and the largest observed drafts in the channel (45 ft). The simulation results are presented graphically in Figure 2-7, and show that the maximum wave height from the calculated pressure field may be as large as 3.5 ft. The zero-crossing half-wave period (drawdown period) for this event is approximately 60 seconds. Herbich et. al. (1976) reported surge heights in excess of 4 ft near the Barge Slip Harbor at the Harbor Island Fabrication Yard.

However, wave amplification was believed to occur at the harbor and therefore these measurements do not represent ambient pressure field wave heights at other locations. For further analysis, a wave height and period of 3.5 ft, 60 seconds were selected as the design criteria.

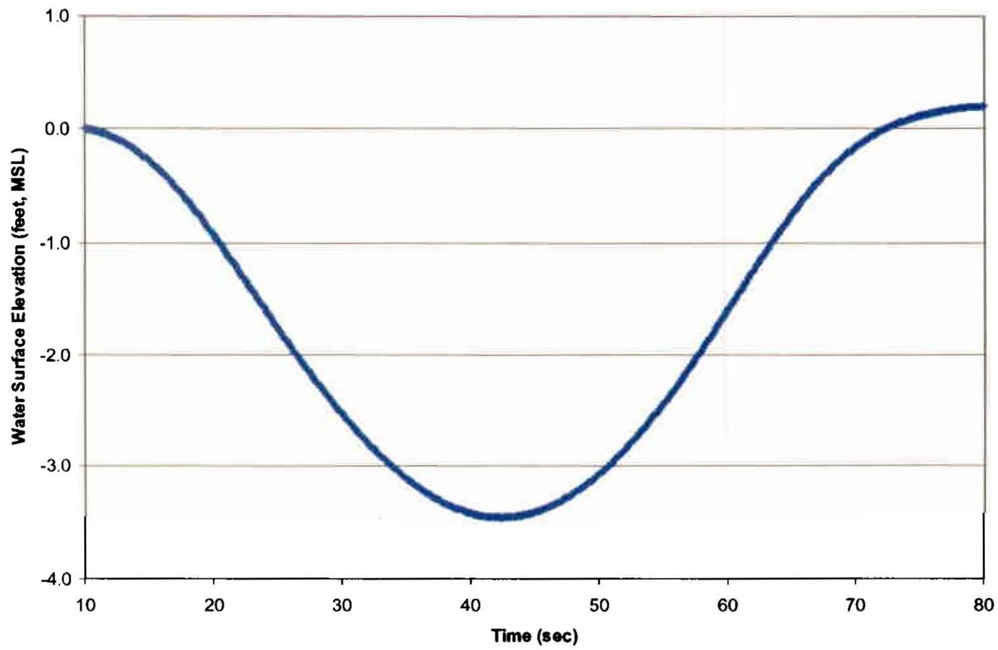

Figure 2-7 Predicted Design Pressure Field Wave for Vessel "Algarrobo"

The following is a conversion of the pressure field wave parameters into hydrodynamic effects that can be expected at the shoreline and must be taken into consideration during the design of shoreline erosion protection structures.

2.1.3 Pressure Field Impacts Analysis and Estimates

Pressure field wave propagation from the deep channel design depth of 45 ft toward the shallow bank (depth less than 6 ft) results in the transformation of long-period wave energy into cross-shore motions of a significant mass of water. These motions may consist of several phases:

- Phase 1 - original slight movement onshore;
- Phase 2 - significant outflow toward the channel when the bottom slope can be exposed far from the shoreline;
- Phase 3 – return water flow in a form of a bore that results in a rapid increase in water level; and,

- Phase 4 - propagation of short-period waves on top of the return flow.

It should be noted that occurrences of the above phases of motion depends on the pressure field parameters. For small pressure field waves, some phases may be so insignificant that they cannot be noticed by the observer or by standard measuring equipment.

The pressure field effects and forces created by motions of the pressure field wave on the shallow bank are investigated for Phase 3 in terms of wave run-up and bottom flow velocities. It is assumed that this phase generates the maximum forces and impacts the shoreline and shoreline structures.

2.1.4 Pressure Field Long-Period Wave Runup/Return Flow Impacts

Pressure field long-wave impacts were investigated using RBREAK2 (Kobayashi and Poff, 1994), a 1-D time domain numerical model for simulation of wave transformation, runup and overtopping.

RBREAK2 is based on nonlinear shallow water wave equations, and is hence appropriate for simulating long-period waves such as pressure field waves. RBREAK2 also allows input of water surface elevation at the seaward boundary. The objective of the modeling was to obtain an estimate of bore speed and height prior to impacting the bluff to aid in designing erosion protection measures.

RBREAK2 allows measurement of water surface elevation at multiple locations in the domain and measurement of the entire water surface profile as a function of time. In order to determine the bore speed at the bluff, the design vessel "Algarrobo" was used to generate an input water surface elevation time series (pressure field wave at deep water of 3.5 ft in height, 60 seconds in period, see Figure 2-7). The pressure field wave propagated up the profile towards the bluff location during the return flow portion of the process.

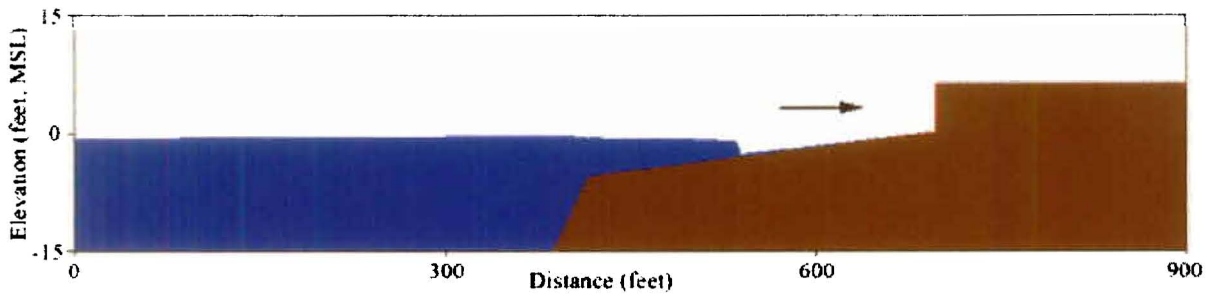

Figure 2-8 Return Flow Bore (Long-Wave Runup) Halfway to Bluff (Simulation Time t=88.8 seconds)

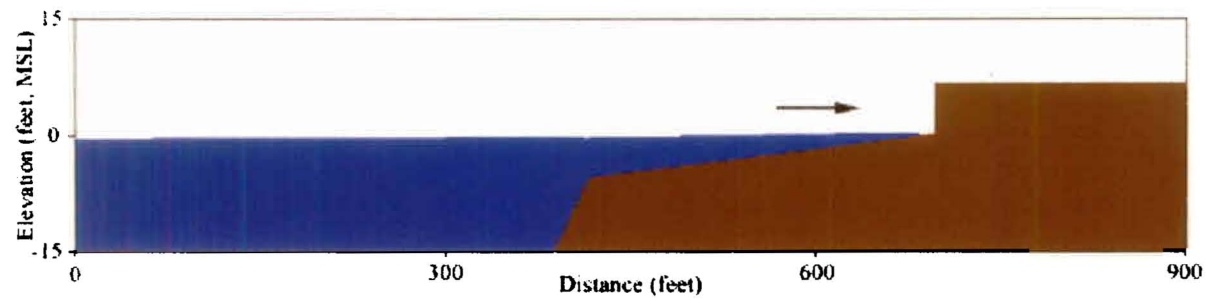

Figure 2-9 Return Flow Bore (Long-Wave Runup) Prior to Bluff Impact (Simulation Time t=103.6 seconds)

Figure 2-8 shows the water surface elevation profile during the point when the return flow has reached halfway between the channel and the bluff. The bore speed is approximately 2 ft per second at this location. Figure 2-9 shows the water surface elevation profile just before the bore would hit the bluff. At this point, prior to impact with the bluff, the bore speed has increased to approximately 8 ft per second.

The RBREAK2 modeling has shown that the long-period wave runup bore can reach speeds up to 8 ft/sec prior to impacting the beach and vertical bluff. Considering that most of the unprotected shorelines along the CCSC consist of fine to medium sand, this velocity most likely significantly contributes to existing erosion. Design measures for erosion protection should be able to withstand the broken long-waves (bores) at approximately 1 ft in height impacting the bluff at roughly 8 ft per second on a daily basis.

2.2 Tidal Currents Impacts

Potential impacts from tidal currents in the CCSC have been studied based on data from the Texas Water Development Board (TWDB) study.

The TWDB collected Acoustic Doppler Current Profile (ADCP) data for several cross-sections along the CCSC between May 5th and 7th, 2000. The data relevant to the project were collected in the vicinity of the University of Texas bulkhead, approximately 1,500 ft east of the Port Aransas ferry terminal. The location of the transect where data were collected is presented in Figure 2-10. The ADCP was mounted on a vessel that collected current velocities across the channel from the southern shoreline of Harbor Island to the northern shoreline of Mustang Island. The data collection effort coincided with a spring tidal cycle to ensure that maximum flows under normal conditions would be measured.

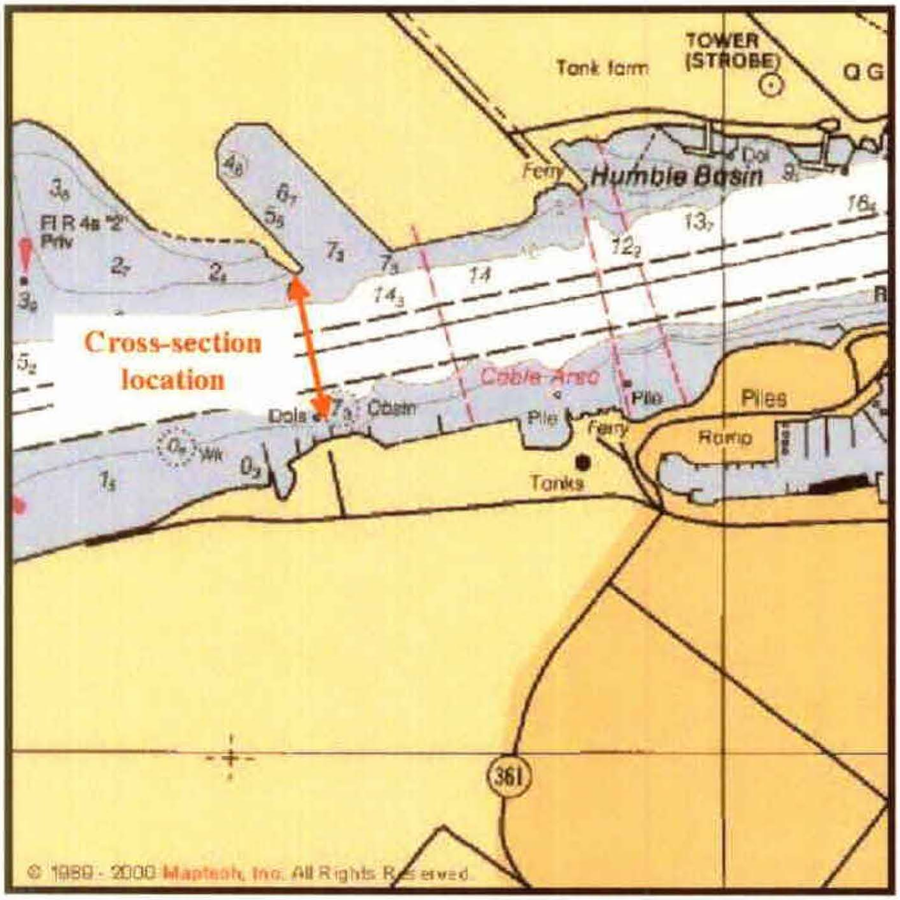

Figure 2-10 Location of Transect Where Data Were Collected

The example of measured and processed data is presented in Figure 2-11 (Prepared by the TWDB). The maximum current velocities measured at the project were recorded near the water surface in the middle of the channel. These velocities ranged between 5 to 6 ft/sec. Away from the center of the channel, the velocities decreased to a minimum at the shallower depths close to the shorelines. The minimum depth at which the current velocity data was collected was at the northern shoreline of Mustang Island, at a depth of 5.3 ft. The measured velocity at this depth was 0.37 ft/sec, which was measured during maximum flow. Most of the lowest-depth measurements taken at the northern shoreline of Mustang Island were taken at depths of 7.0 ft. The highest current measured at this depth and location during maximum flow was 1.51 ft/sec.

The preliminary analysis indicates the following:

- Current velocities capable of eroding bottom sediment are observed only in the channel cut. Current velocities decrease significantly with the distance from the channel cut. Current velocities outside of the channel are relatively small and do not significantly contribute to the shoreline erosion.
- Since velocity transects were measured during a spring tide, it is likely that the velocity gradient is close to a maximum. Weaker gradients and slower maximum flows would be expected under neap tide conditions.
- It is unlikely that tidal current velocity alone causes or contributes significantly to shoreline erosion along the CCSC.

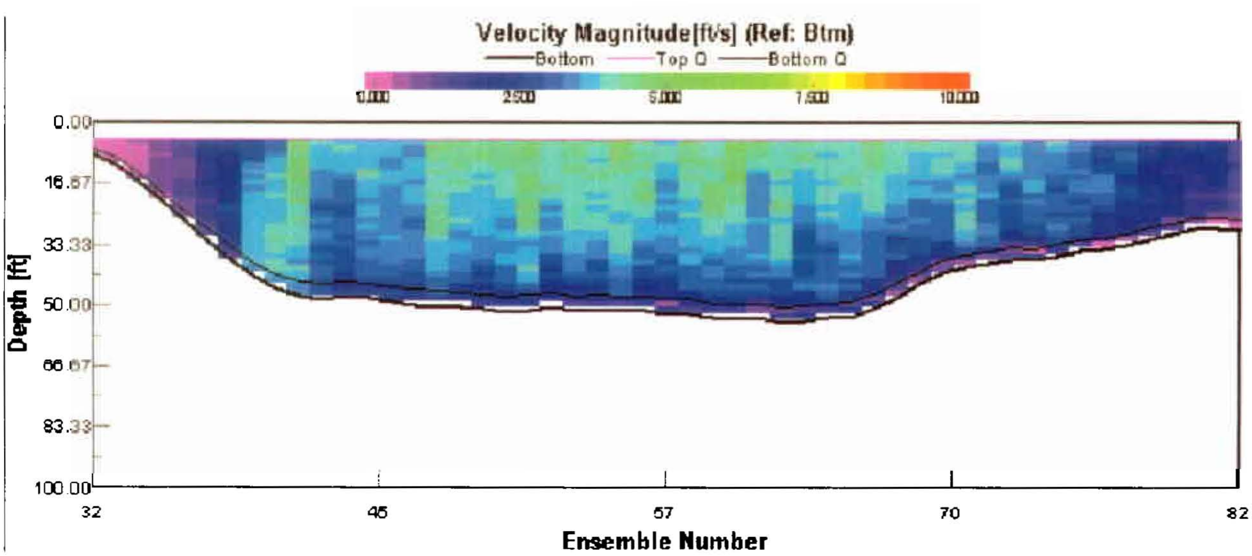

Figure 2-11 Current Velocity Distribution Across the CCSC (Selected Cross Section)
Source: TWDB

3. Watermain Shoreline Stabilization Alternatives

3.1 Introduction

Shoreline stabilization alternatives were developed for each of the 5 areas of concern. These areas of concern were established by the GLO, the City of Port Aransas, and the Port of Corpus Christi Authority, to be: 1) Waterline Crossing, 2) Access road to fishing pier 3) Public park shoreline, 4) Shoreline between Piper Channel and the public park, and 5) Harbor Island. An alternatives analysis has been performed for each of the areas of concern. However, this report presents the results of the alternative analysis for the Watermain Project only. The alternatives analysis for the other areas of concern will be provided in subsequent amendments to this report. The analysis of the alternatives was conducted using the following evaluation criteria:

- Construction Cost
- Maintenance Requirements
- Shoreline protection performance
- Reduction of the loss of property and public infrastructure
- Enhancement of public area usability and accessibility for recreational purposes
- Preservation and enhancement existing coastal wetlands and habitats
- Reduction of shoaling within navigational channel.

A summary of the results from the alternatives analysis is presented in Table 3-1. Three sets of assumptions were used during the development of the alternatives analysis: Cost estimate assumptions, miscellaneous engineering criteria assumptions, and construction material assumptions.

Construction Cost Estimate Assumptions

Construction costs estimates are based on the following assumptions regarding unit costs for materials:

- Bulk backfill - $12.00 per cubic yard (cy)
- Granular backfill - $15.00 per cy ($5.00 cy material cost, $6.00 cy transportation, $4 cy placement)
- Armor rock- $68 per ton

- Core/bedding rock for groin - $50.00 per ton
- Shoreline placed Geotextile tube -$250 per linear ft
- Submerged placed concrete Armorflex mat - $30 per square ft
- Shoreline placed concrete Armorflex mat - $15 per square ft
- Concrete bulkhead - $25 per square ft
- All costs are based on in-place construction costs
- Construction costs for all alternatives were increased by 25% as a contingency, due to unknown factors

Considering the amount of assumptions used, the presented cost estimate should be considered rather as a relative for the alternative analysis. If assumptions change, the cost estimate for each of the alternatives may change dramatically, however, the relative cost between the alternatives would be approximately the same.

Construction material costs were obtained from local material suppliers. Construction costs were obtained from local general contractors (W.T. Young Construction, King Isles Construction, Inc., Gulf Coast Lime Stone, Inc.). Armor rock costs were obtained from quarries located in the Georgetown Texas area for land transportation and from Arkansas (River Mountain quarry) for barge transportation. It was assumed that a fill source would be available within a 20-mile radius of the project site.

Miscellaneous Engineering Criteria Assumptions

Preliminary miscellaneous engineering criteria assumptions were developed to address the following three issues: Shore protection structure crest elevation, potential scour hole, and size of armor rock and Armorflex.

Shore protection structure crest elevation (bulkhead, jetty, or revetment) of + 7 ft mlt.

This criteria has been derived from the following considerations: Concrete bulkheads have been used extensively along the ship channel as shoreline protection measures for 30 to 40 years. The top elevation of these bulkheads ranges from elevation +5 to +7 ft mlt. Hurricanes and storms during this time period resulted in little or no overtopping (Urban, 2000). Extreme surge elevation events (USACE, 1984) were used to select the crest elevation design criteria. The USACE study shows that the Corpus Christi upper bay extreme surge elevation of a 20-year return period is approximately +6.4 ft NGVD, or +7.2 ft mlt. Assuming that a return period of 20 years is a design criteria, the preliminary design top elevation is selected at +7 ft mlt. All alternatives were evaluated using these criteria.

Potential scour hole depth in front of the structure

It has been assumed that the maximum depth of scour would be equal to half the long-period wave height or half the return flow thickness. This results in scour depths of 0.5 to 1.0 ft on the upper beach above 1.0 ft, MLLW, and scour of 1.0 to 20 ft on the lower beach between + 1.0 and –5.0 ft, MLLW.

Size of the armor rock, Armorflex dimensions

The hydrodynamic analysis presented in Section 2 determined that long-period pressure field waves are a critical factor in shoreline erosion. The specification of the maximum armor stone size is based on two approaches:

1. Consideration of height and period for the design long wave; and,
2. Consideration of return flow velocities for the design long wave.

The first approach yields a minimum weight of 2,200 lbs, while the second approach yields a limit of 2,500 lbs.

Construction Material Assumptions

- The sheet-pile bulkhead could be constructed using either concrete or vinyl material. Concrete is readily available, inexpensive, and has been successfully used for various projects along the CCSC at the City of Port Aransas shoreline. Vinyl sheetpile walls may be less expensive alternative. However, less experience exists in its application for local projects. The final decision of material to be used shall be made during the final engineering design. At this stage of the study, it is assumed that the bulkhead is constructed with concrete.

- None of the evaluated alternatives utilize concrete articulated mats such as "Armorflex" as a scour protection material. It was determined that the construction cost for the Armoflex protection for underwater installations is at the same cost range as the cost of using rock protection. In addition, concerns about the performance of the Armoflex in long-period wave environment still exist due to the lack of available information and practical experience in similar types of environments. If it is determined during the final engineering design that the Amorflex has been used successfully in the same wave environment, it could then be used in lieu of the rock protection.

3.2 Watermain Project Alternatives

Five shoreline erosion stabilization alternatives for the Watermain Project have been developed and are discussed below. The basis for developing each of the alternatives as different and included the following:

Alternative 1 is based on the protection concept presented in the May 8, 2000 Shiner & Moseley Associates, Inc., U.S. Army Corps of Engineers permit application. This concept was modified by PI Engineering to result in an alternative with the same level of performance and consistency as the other alternatives, as identified during the Task 1 analysis (see Project Review Report -Appendix B).

Alternatives 2 through 5 are based on the results of the causes of erosion analysis (see Section 2) and based on PI Engineering experience with similar shoreline erosion protection projects.

Alternative 6 is based on the configuration proposed by Urban Engineering with modifications to achieve the same level of performance and consistency as the other alternatives for the alternatives analysis.

3.2.1 Alternative 1 – Full Length Bulkhead

Alternative 1 consists of a sheet-pile bulkhead along the existing shoreline (approximately 0.0 ft mlt) extending from the TAMU bulkhead to the ferry landing facilities. The area behind the bulkhead would be filled with imported fill materials. A plan view of Alternative 1 is presented in Figure 3-1. A bulkhead crest elevation of +7 ft, mlt was selected based on the design criteria assumptions (see Section 3.1) to match the top elevation of the existing TAMU bulkhead. The area behind the bulkhead is filled with two types of fill material: granular material along the immediate area behind the bulkhead and generally less expensive fill further back from the wall.

At the intersection of the bulkhead with the pipeline, the bulkhead has a gap filled with armor rock as shown in Figure 3-1, Section A-A. This gap is to preclude damage of the pipeline during the placement of the concrete bulkhead. For this alternative, this measure is required because the pipeline is embedded approximately 2.0 ft. The gap in the bulkhead is filled with the armor rock placed on the bedding layer.

The pipeline extension from the bulkhead toward the channel is covered with armor rock underlayed by the bedding layer to prevent erosion in front of the bulkhead and exposure of the pipeline (Figure 3-1, Section B-B).

Alternative 1 is derived from the originally proposed measure by the City of Port Aransas as presented in the permit application prepared by Shiner & Moseley Associates, Inc. The current Alternative 1 differs from the previous permit application alternative such that the current alternative:

- Requires an intersection between the bulkhead and pipeline.
- Provides a bulkhead toe to be submerged below the potential depth of scour.
- Provides a crest elevation at +7 ft for the entire length of the bulkhead, while the Permit Application specifies part of the bulkhead at an elevation of +3 ft mtl.
- Provides a straight bulkhead alignment rather than multidirectional. Straightening of the bulkhead is proposed due to the potential for scour at the bulkhead corners.

The size of the armor rock to protect the pipeline is selected based on the analysis of hydrodynamic forces acting on the rock (see Section 2). The maximum weight armor rock for this structure is estimated at 2,200 lbs.

A summary of the Alternative 1 evaluation is presented in Table 3-1.

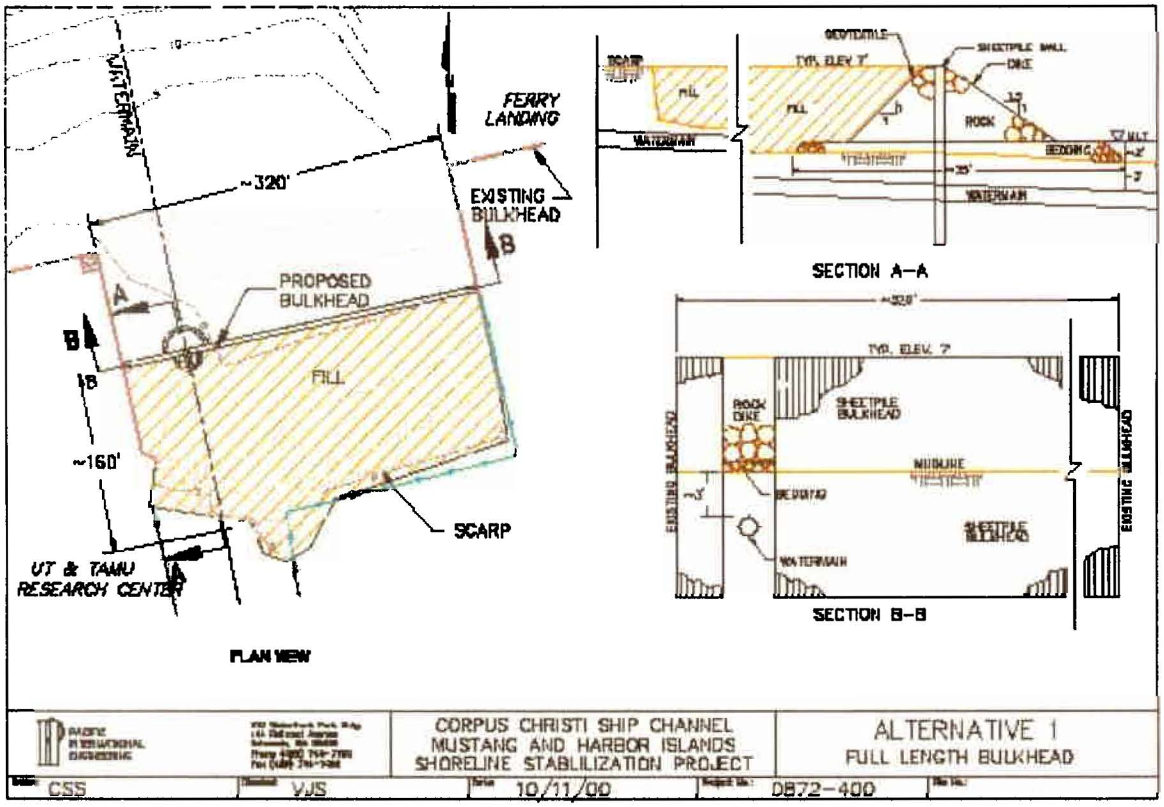

Figure 3-1 Plan View of Alternative 1

3.2.2 Alternative 2 – Partial Length Offshore Bulkhead

Alternative 2 consists of a new sheet-pile bulkhead extending from TAMU bulkhead, a distance of 100 ft parallel to the existing shoreline to Corner Point. From Corner Point, the new wall would turn 90 degrees and extend in the onshore direction to the existing shoreline. The nearshore section of the bulkhead keys into the existing bluff. A plan-view of Alternative 2 is presented in Figure 3-2. Alternative 2 bulkhead connects with the TAMU bulkhead at an approximate depth of –1.0 ft mlt that is approximately 30 ft inland from the tip of TAMU bulkhead (or approximately 150 from the shoreline).

A crest elevation of +75 ft mlt is proposed for the bulkhead to match the existing TAMU bulkhead at an elevation of +7. The area behind the bulkhead is filled up to the top of the bulkhead with two types of fill material: immediately behind the bulkhead – granular material fill, all other – less expensive locally available fill.

Alternative 2 proposes toe protection for the bulkhead and pipeline. A cross section along the pipeline depicting the intersection of the bulkhead with toe protection and the pipeline is presented in

Figure 3-2, Section A-A. The toe protection consists of a layer of armor rock placed on a bedding layer. The armor rock was sized based on the analysis of hydrodynamic forces acting on the rock (see Section 2). The maximum weight of armor rock for toe protection is estimated at 2,200 lbs.

If required, a stormwater drainage system may be embedded into the fill with outfall at the bulkhead face. The need for a drainage system in Alternative 2 and drainage elements specifications (if required) should be evaluated during the final engineering design phase.

A summary of the Alternative 2 evaluation is presented in Table 3-1.

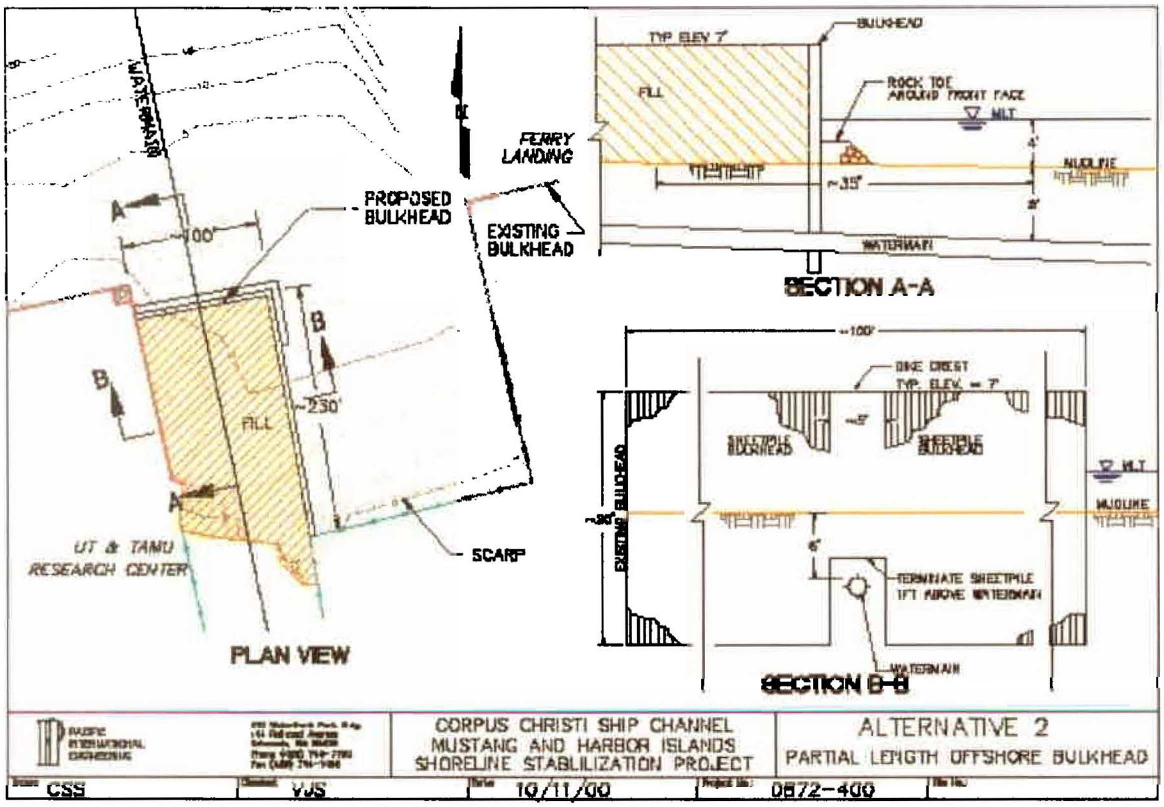

Figure 3-2 Plan View of Alternative 2

3.2.3 Alternative 3 – Gravel-Rock Terraces

Alternative 3 consists of two parallel rock dike terraces with coarse gravel and sand fill. A plan-view of Alternative 3 is presented in Figure 3-3. The crest elevation of the lower terrace is at +2 mlt, while the crest elevation for the upper terrace is at elevation +7.0 mlt (similar to that of TAMU bulkhead). Cross sections of the terraces along the pipeline are presented in Figure 3-3, Sections A-A and B-B.

The size of the rock required for the dike and gravel fill is selected based on the analysis of hydrodynamics forces (see Section 2.1.1.) and previous experience with the stepped shoreline erosion protection structures. The evaluation of rock size is based on the premise that the first step splits the wave energy allowing part of it to reflect and the remainder to propagate toward the second step. Therefore, the impact from wave energy on the dike is reduced and the size of the rock required is less than for an equivalent dike without a step.

If required, the drainage system may be embedded into the steps with an outfall at the face of the dike. The need for a drainage system in Alternative 3 and drainage element specifications (if needed) should be evaluated during the final engineering design phase.

Stepped rock terraces will dissipate erosive wave forces incrementally as waves move across the structure profile. Wave energy will not be blocked, dissipated, or reflected at a single location, but rather across the structure's terraced profile. This type of wave energy reduction helps to reduce problems of scour at the toe of the structure, wave overtopping, and wave reflection that are associated with vertical hard structures. The lower rock terrace will be aligned with the TAMU wall to the northwest, to minimize "corner effects" of irregularities in the water's edge, and to maximize the reclaimed upland area. The crest of the lower rock terrace will be at elevation +2 ft mlt, and retain a 100-ft-wide strip alongshore of stable granular material suitable for a recreational area. The toe elevation of the lower rock terrace accounts for one foot of structure-induced scour and two feet of bottom lowering during the project life

A summary of the Alternative 3 evaluation is presented in Table 3-1.

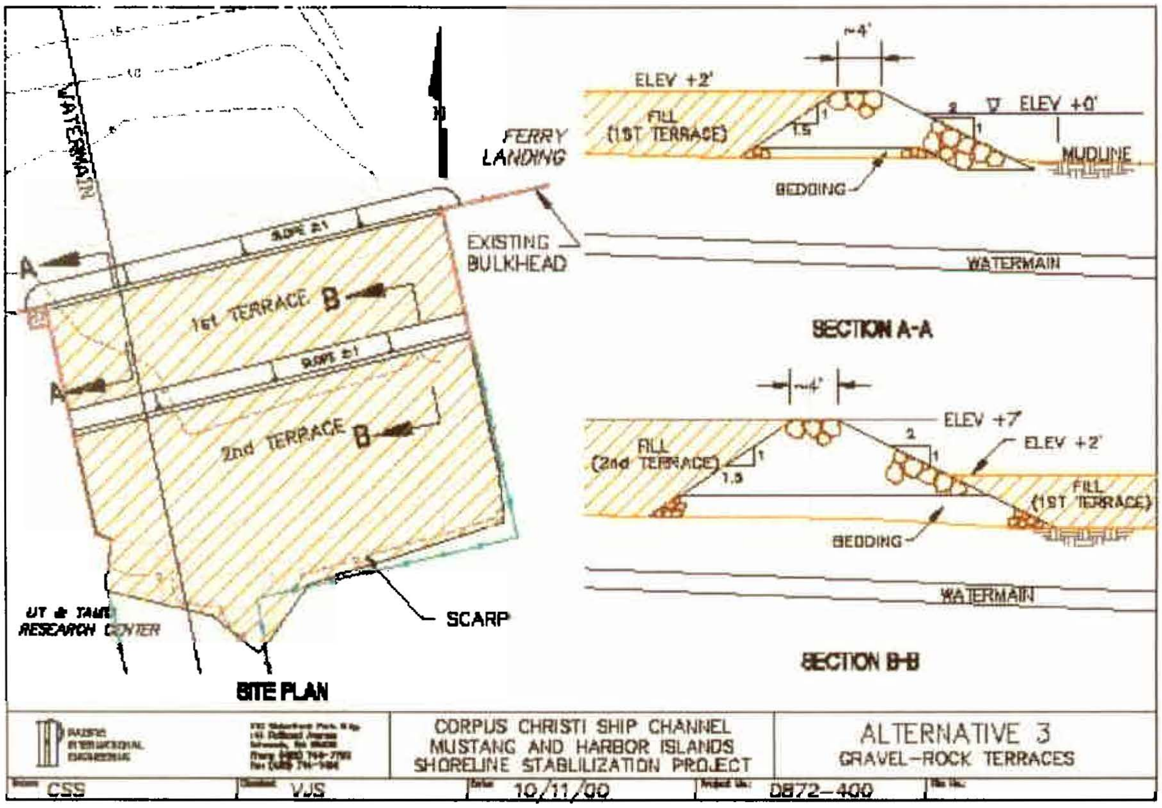

Figure 3-3 Plan View of Alternative 3

3.2.4 Alternative 4 – Jetty and Pocket Beach

Alternative 4 consists of a jetty placed parallel to the shoreline, extending from the TAMU bulkhead to the Ferry Landing Facilities with an opening in the middle. The inland area behind the jetty is formed as a pocket beach. A plan-view of Alternative 4 is presented in Figure 3-5.

It can be seen that the toe of the jetty aligns with the seaward face of the TAMU bulkhead. This is to provide protection for the failing corner of the bulkhead. However, if it is not an issue for the project and to avoid interference with the deep scour hole at the tip of TAMU bulkhead, the jetty may be offset approximately 30-40 ft landward from the tip of the TAMU bulkhead. In this case, the amount of material for the project (rock, sand, gravel) would significantly reduce the costs for this alternative. A decision regarding offsetting the jetties should be made by the Project Owner, in coordination with the GLO and the City of Port Aransas. At this stage of the analysis, the Alternative 4 cost estimates assumptions are conservative in assuming that the jetties are not offset.

The crest elevation of the Jetty is at approximately +7.0 ft mlt (similar to the TAMU bulkhead). The width of the opening for this preliminary design is estimated at 60 ft. Longitudinal cross section through the jetty is presented in Figure 3-5, Section C-C.

The opening in the jetty is to allow limited wave propagation toward the pocket beach to maintain water quality and appropriate habitat. A scour protection rock blanket is proposed in the opening to prevent potential scour due to high velocities between the jetties. Cross sections through the opening and the jetty are presented in Figure 3-5, Section B-B and Section A-A, respectively.

The size of the opening controls the amount of wave energy which may penetrate inside and impact the beach, and also the bottom velocities and the scour in the opening itself. Preliminary sensitivity analysis using numerical modeling of the flow penetrating through the opening and distribution of flow velocities inside the pocket beach has been completed using RMA-2D model. The results of the modeling were used to select the optimum size of the opening and to supplement the decision regarding the size of the rock required to prevent scour in the opening. An example of the modeling results are presented in Figure 3-4. Figure 3-4 presents the velocity distribution of three scenarios: A - without gap of 50 ft, B – gap of 50 ft, and C - gap of 100 ft. It can be seen that increasing the gap width reduces velocity in the gap. Altering the width of the gap in the model, if proposed as a design criteria, is recommended at a width of approximately 60 ft.

The pocket beach behind the jetty is built from imported sand and limited amount of gravel. Gravel is placed at the top of sand only at the area in front of the opening. The length of the gravel beach equals 1.5-2 length of the opening that yields approximately 90-120 ft.

The size of the rock for the jetty is selected based on the analysis of hydrodynamics forces (see Section 2.1.1) and previous experience with rock jetty construction. The maximum weight of armor rock for the jetty is estimated at 2,500 lbs.

It is assumed that a drainage system is not required for Alternative 4. A summary of the Alternative 4 evaluation is presented in Table 3-1.

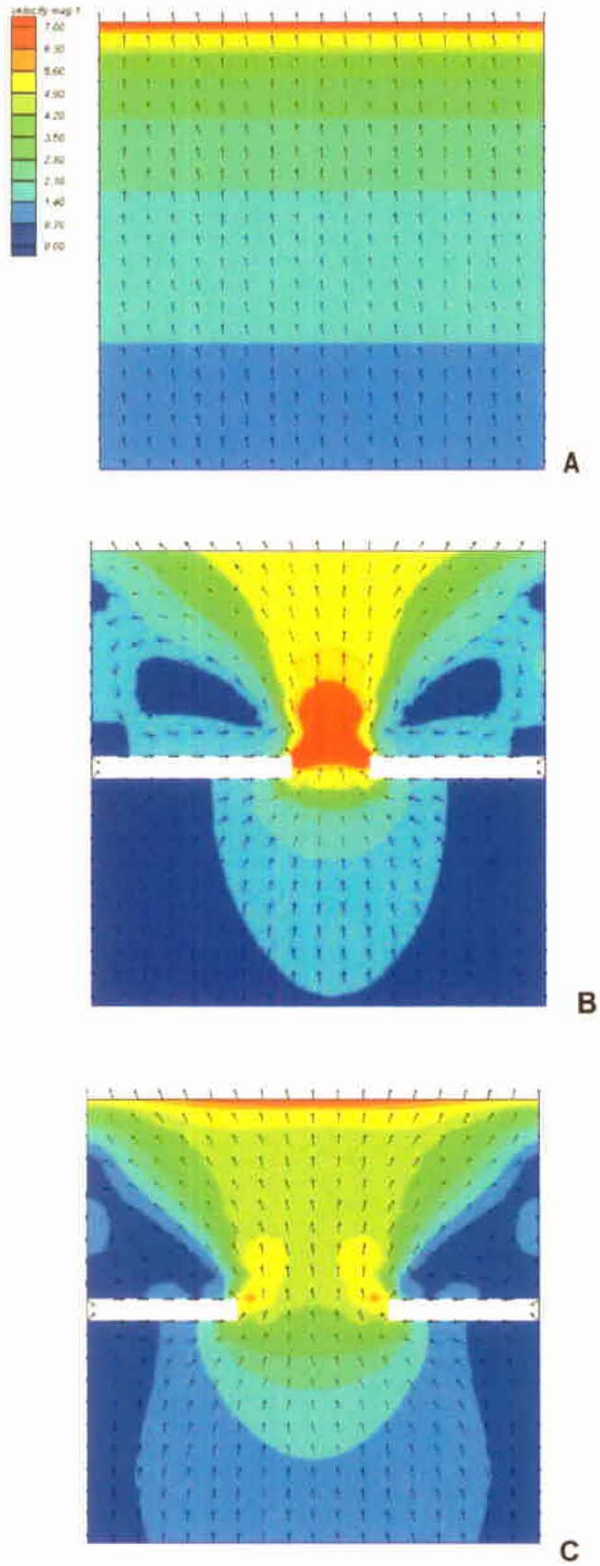

Figure 3-4 Gap Width Sensitivity Modeling (Example) Results

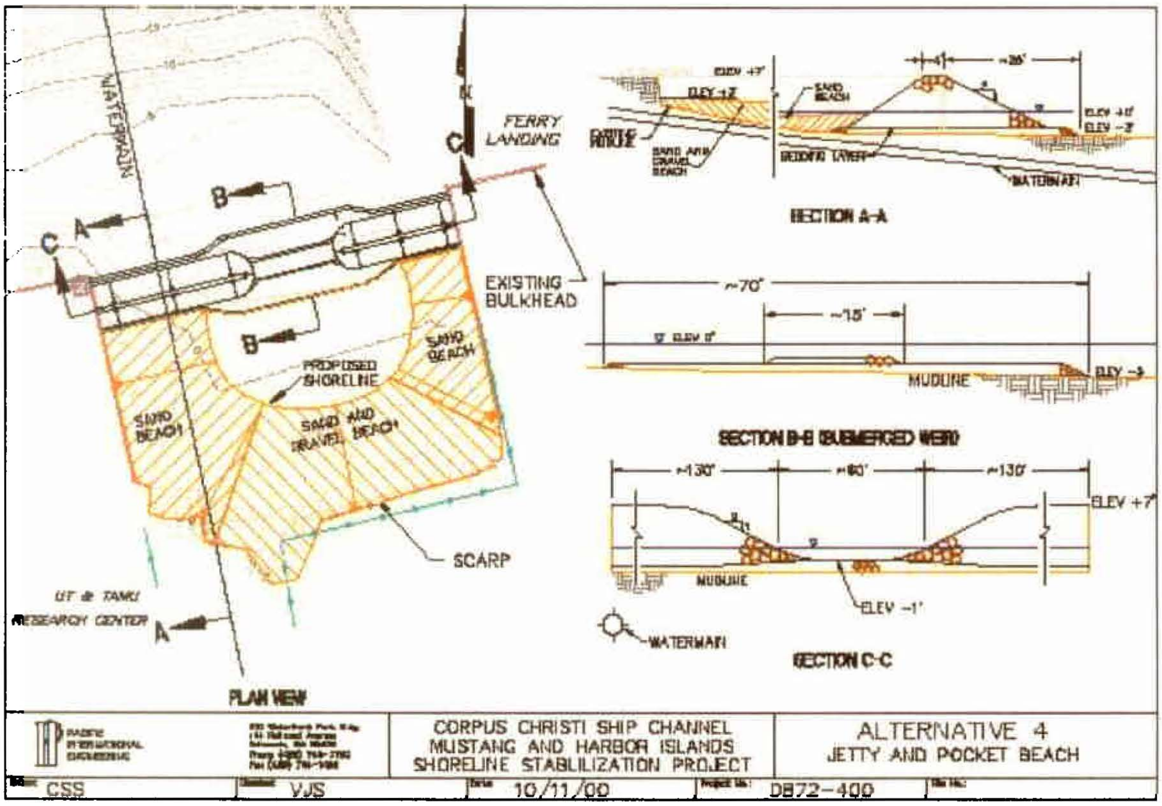

Figure 3-5 Plan View of Alternative 4

3.2.5 Alternative 5 – Pipeline Armor Rock Revetment

Alternative 5 consists of an armor rock revetment with a bedding layer placed along the pipeline alignment to prevent potential scour and exposure of the pipeline. A plan-view of Alternative 5 is presented in Figure 3-6.

The crest elevation of the armor rock varies along the pipeline and is parallel to the beach bottom slope. At the landward end of the protection, crest elevation is approximately +7 ft mlt. At the seaward end of the protection the crest elevation is approximately +4 ft. Cross sections along the pipeline and perpendicular to the pipeline are presented in Figure 3-6, Section B-B and A-A, respectively.

The size of the armor rock required for the pipeline protection is selected based on the analysis of hydrodynamics forces (see Section 2.1.1). The maximum weight of the armor rock is estimated at 2,200 – 2,500 lbs.

A drainage system is not required for Alternative 5. A summary of the Alternative 5 evaluation is presented in Table 3-1.

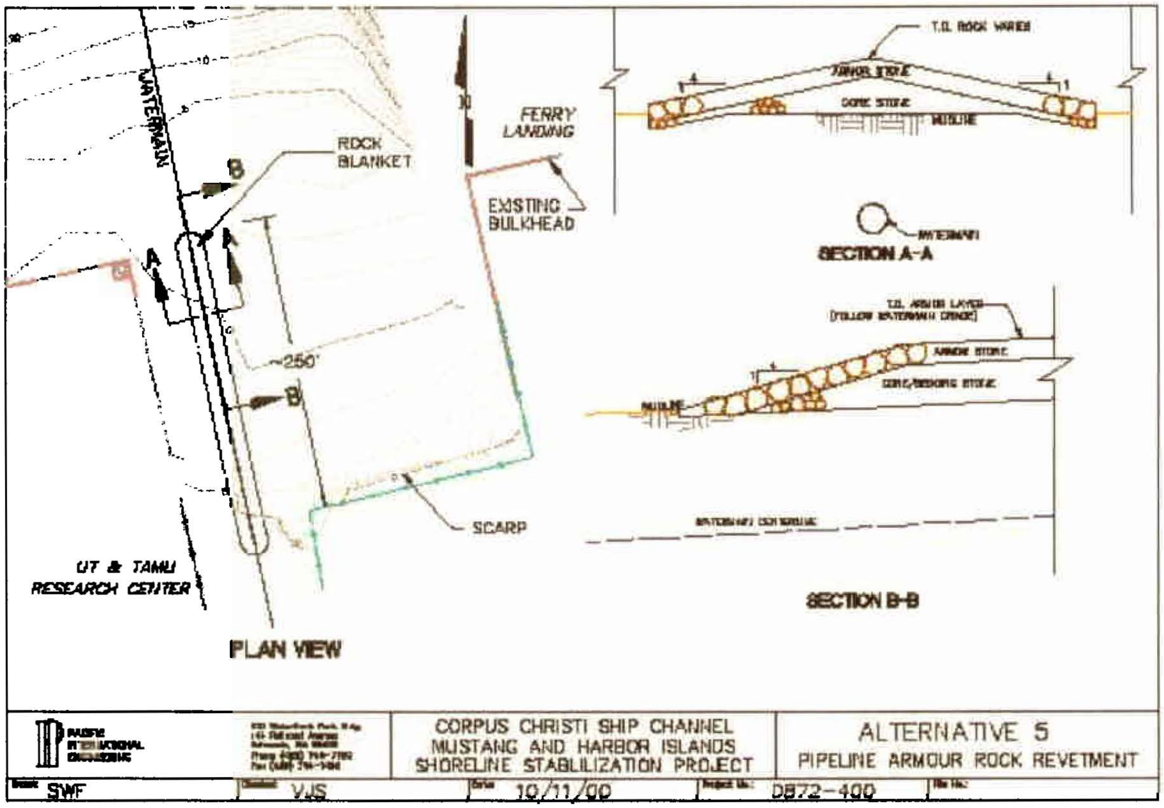

Figure 3-6 Plan View of Alternative 5

3.2.6 Alternative 6 – Offshore – Nearshore Bulkhead

Alternative 6 was prepared by Urban Engineers and modified by PI Engineering to bring this alternative to the same level of performance and consistency in the alternatives analysis. This alternative consists of a new sheetpile bulkhead and armor rock scour protection to be constructed between the TAMU bulkhead and the ferry terminal bulkhead. A plan-view of alternative 6 is shown in Figure 3-7.

The intersection of the new bulkhead and the existing TAMU bulkhead is at an approximate depth of 0.0 mlt, or approximately 80 ft waterward of the existing TAMU concrete bulkhead terminus. The bulkhead alignment would extend parallel to the shoreline for a distance approximately 75 ft from the TAMU bulkhead and then angle back to the existing shoreline near existing elevation +2 ft. The bulkhead would then extend along the shoreline to the southeast corner of the project site and then waterward along the eastern project boundary to the intersection with the existing ferry terminal concrete bulkhead. The existing timber bulkhead would be demolished.

The new bulkhead was located near mean low tide at the intersection of the TAMU bulkhead in order to provide sufficient protection at the waterline. The existing soil cover over the pipeline at this location was estimated to be at least 6 ft. This provides adequate distance to obtain some embedment depth for the bulkhead and also for the installation of pipeline erosion protection. The bulkhead would utilize pre-cast concrete sheetpiling with tiebacks for the entire project except at a 10 to 15 ft width over the watermain. This location would require a cast-in-place concrete wall keyed into the concrete sheetpiling. Extended length sheetpiling would be placed adjacent to the watermain wall section to resist additional lateral loading resulting from the reduced embedment depth along the cast-in-place section of bulkhead. The top of bulkhead would be +7 ft along the entire length of the bulkhead. Stairs could also be integrally constructed into the concrete sheetpile wall to provide access to the beach. The cost of the stairs was not included in this alternative.

A summary of the Alternative 6 evaluation is presented in Table 3-1.

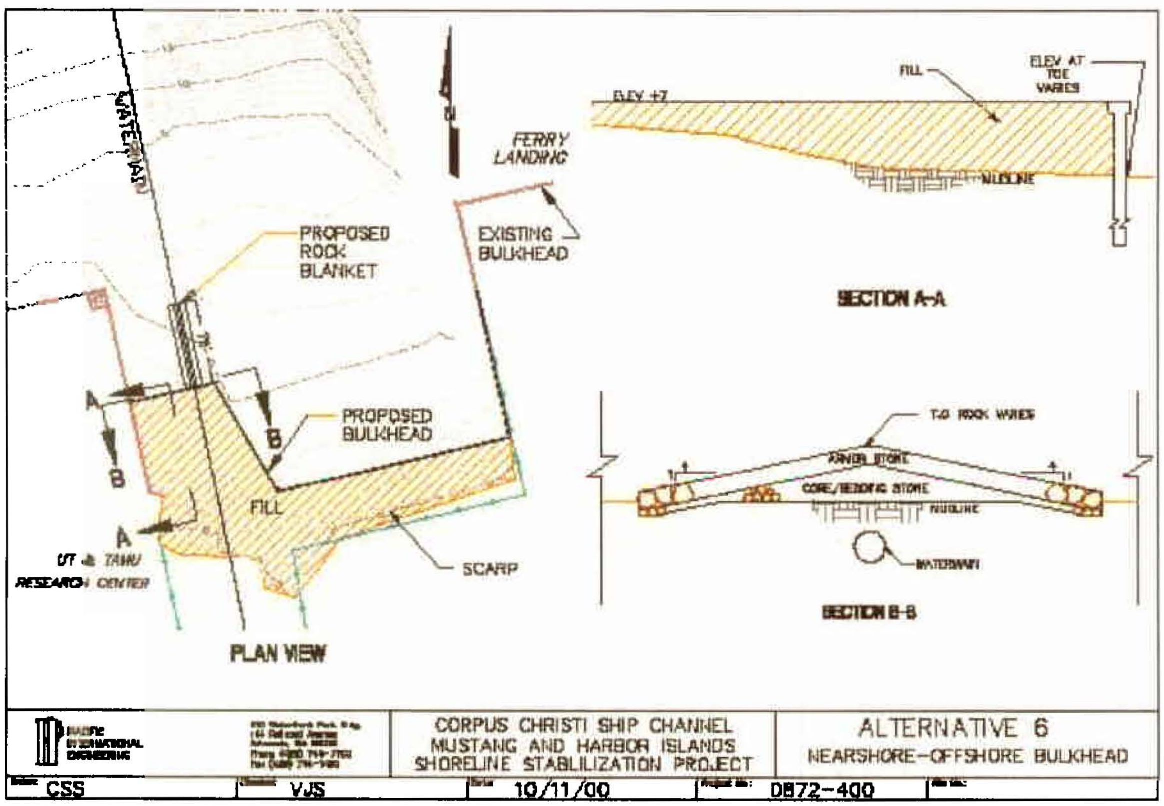

Figure 3-7 Plan View of Alternative 6

3.2.7 Summary of Waterline Crossing Alternatives Analysis

The criteria for analysis and selection of the preferred alternative were specified by the Scope of Work for the study and are discussed in Section 3.1 of this report. The proposed alternatives were evaluated with regard to these criteria and brief results of the evaluation are presented in Table 3-1. Using these evaluations, a preliminary ranking in order of preference of the alternatives has been developed as follows:

1. Alternative 4–Jetty and Pocket Beach
2. Alternative 6–Offshore-Nearshore Bulkhead
3. Alternative 1–Full Length Bulkhead
4. Alternative 5–Pipeline Armor Rock Revetment
5. Alternative 3–Gravel Rock Terraces
6. Alternative 2–Partial Length Offshore Bulkhead

Considering that the alternatives evaluation is based on a significant amount of assumptions, it is recommended that the selection of a preferred alternative be conducted in coordination with the GLO, the City of Port Aransas, and the Project Owner.

Table 3-1 Summary of Alternatives Analysis

Corpus Christi Ship Channel – Watermain Erosion Protection Alternatives Analysis							
Alternative	**Initial cost estimate***	**Maintenance requirements**	**Shoreline protection performance (loss of public areas)**	**Watermain protection performance (loss of infrastructure)**	**Accessibility and usability for recreational purposes**	**Reduce shoaling within the navigation channel**	**Preserve existing coastal wetlands and habitat**
1 Full Length Bulkhead	$325,000-$400,000	Maintenance for rehabilitation of armor rock along the pipeline one time per 25-year project life at a cost of $20,000.	Reliable for the shoreline and landward infrastructure between TAMU and Ferry Terminal bulkheads.	Reliable for existing conditions. Must be re-evaluated for channel deepening conditions.	No pedestrian or car access to the beach. Precludes beach recreational use, but reclaimed land has potential for recreational use.	No impact	Reduces shallow water habitat.
2. Partial Length Offshore Bulkhead	$290,000-$360,000	Maintenance for rehabilitation of armor rock along the pipeline one time per 25-year project life at a cost of $20,000	Limited. Area without bulkhead is at risk for continued erosion.	Reliable	Provides beach access. Reduces recreational value of the beach, but increases the potential of land recreational use	No impact.	Reduces shallow water habitat.
3. Gravel Rock Terraces	$550,000-$700,000	Maintenance, would include rehabilitation of fill, is assumed to be $130,000 for the 25-year project life.	Reliable for the shoreline and landward infrastructure between TAMU and Ferry Terminal bulkheads.	Reliable	Provides water access (gravel / rock beach) but will reduce sandy beach access. Has a potential of increasing the recreational use of land.	No impact	Will slightly reduce shallow water habitat, but will create habitat diversity.

Corpus Christi Ship Channel – Watermain Erosion Protection Alternatives Analysis							
Alternative	**Initial cost estimate***	**Maintenance requirements**	**Shoreline protection performance (loss of public areas)**	**Watermain protection performance (loss of infrastructure)**	**Accessibility and usability for recreational purposes**	**Reduce shoaling within the navigation channel**	**Preserve existing coastal wetlands and habitat**
4 Rock Jetty and Pocket Beach	$320,000-$400,000	Would include rehabilitation of the armor rock 1 time per, 25-year project life at a cost of $25,000.	Reliable for the shoreline and landward infrastructures between TAMU and Ferry bulkheads.	Reliable	Provides pedestrian and cars beach access. Increases recreational use for the beach and land.	No impact.	Enhances the shallow water habitat area.
5 Pipeline Armor Rock Revetment	$70,000-$100,000	Would include rehabilitation of armor rock 1 time per 25-year project life at a cost of $60,000.	No protection. May increase erosion.	Reliable	No change from present beach access condition, but may reduce the recreational use of the beach.	No impact.	Slightly reduces existing habitat area.
6 Offshore–Nearshore Bulkhead	$330,000 - $415,000	Would include placement of rock at the scour hole at the bulkhead and rehabilitation of the pipeline rock cover one time per 25-year project life at a cost of $35,000.	Reliable for the shoreline and landward infrastructures between TAMU and Ferry bulkheads.	Reliable	Limited pedestrian access and no car access to the beach. Reclaimed land has potential for recreational use.	No impact.	Slightly reduces existing habitat.

*Note: The initial cost estimate does not include the cost of permitting a particular alternative. It is assumed that a Section 10 U.S. Army Corps of Engineers Permit and a Section 404 Clean Water Permit will be required at the Federal level. Further examination of State and local permitting requirements will be conducted by the appropriate entity following selection of the preferred alternative.

4. References

A Preliminary Report Upon Cayo Del Oso and Corpus Christi Beach, Prepared for the Area Development Committee, Harland Bartholomew and Associates, September 1966.

Coastal Processes Assessment For Dredging Requirements Reduction At the Piper Channel Entrance, Port Aransas, Texas, Prepared by Kraus and Associates Technical Report 94-1, January 1994.

Department of the Army, Corpus Christi Ship Channel, Texas, Reconnaissance Report, September 1994.

Galveston County Comprehensive Gulf Shoreline Erosion Response Plan, Prepared by Shiner, Moseley and Associates, Inc for Galveston County Beach Erosion Task Force,
January 2,000.

Nueces County Water Control and Improvement District #4, 12" Supply line, Reagan & McGaugham Engineers, August 1968.

Point Park Bulkhead, Port Aransas, Texas, The Construction Plans, Urban Engineering, Job # 216100.00.01, October 1988.

Preliminary Analysis on Shoreline Erosion Corpus Christi Ship Channel in Vicinity Piper Channel, Hartman Consulting Group, April 1996.

Shepsis, et al, Port of Oakland Inner Harbor Waterway Design Considerations, Ports '98, Conference Proceedings, Volume Two, pp. 1268:1276

Surge Attenuation at Barge Slip Harbor at Harbor Island Fabrication Yard, Texas A&M Research Foundation, College Station, Texas, Report No. COE-190, January 1976.

U.S. Army Corps of Engineers, Port Aransas, Texas, Section III, Initial Appraisal, December 1994.

APPENDIX A

Meeting Minutes
July 10, 2000

Corpus Christi Ship Channel Project

Kick-off Meeting

July 10, 2000, 1:00pm

Attendees:

• City of Port Aransas: (CPA)	Tommy Brooks (City Manager) Jim Urban (Consultant/City Engineer)	361.749.4111
• General Land Office (GLO):	Matthew Mahoney	512.475.4591
• PI Engineering:	Vladimir Shepsis Josh Carter Shane Phillips	425.921.1703 512.420.0604 425.921.1709

Meeting Proceedings:

1. Introduction and Background

TB said he is happy to have someone who has experience in this area working on the coastline. He would like this to be a model project for the rest of the CEPRA Program, and wants it to be something that will help the entire TX coastline, not just the CPA. Therefore, he is looking forward to a good quality product that lasts and maybe does not cover the entire area of interest rather than a cheap quick fix that protects the whole shoreline for a just a couple months.

2. Current Project Overview and Goals

VJS gave a presentation on how the project needs to be developed. He stressed that a study was necessary to fully understand the processes involved, especially long period wave characteristics in the channel, which are predicted to be the major cause of erosion in the area.

Presentation by VJS on Project Study

Project Review

Site Visit, Review and Analysis
Permit Application

VJS said permit application is good engineering, and we may use some of their suggestions. However, some may not be feasible, such as geotubes for detached breakwater. However, conclusions on this topic can be developed after completion of the study.

Compilation of Historical Data

JU said that they have conducted a large amount of work in the area. They have available bathymetry, topographic cross-sections, some in digital format. He will provide this information.

Determine Factors Controlling Erosion

Data Analysis
Numerical Modeling
Data Collection, including Topographic, Bathymetric and Geotechnical
Deep draft vessel effects

PI Engineering will deploy instruments for data collection. Some discussion on what should be measured, will most likely include currents, vessel wakes, and pressure fields.

Alternatives Analysis

Develop Range of Alternatives
Analysis and Selection of Preferred Alternatives
Preliminary Engineering and Conceptual Design

TB is very concerned with the waterline. He wants to expedite analysis on the area concerning the waterline for immediate protection. He would like to enter the construction phase ASAP.

VJS responded that it could possibly be conducted more quickly since that specific area is sidelined by two concrete bulkheads.

3. Permitting

SM submitted permit to the COE, with some data supplied by Jim Urban. PI Engineering will review the permit.

Meeting Conclusions (Summary):

1. All parties accepted that the scope of work presented by the GLO and PI Engineering addresses the issues and requirements of the project.
2. All parties agreed to work as a team:
   i. any work will be reviewed and commented on by all other parties
   ii. Coordinate efforts through GLO for best final product for all parties involved. PI Engineering will contact Local Sponsor through GLO (and vice versa), but PI Engineering can contact Local Sponsor directly in search of information. A notice to GLO must be made of all contact.
3. The City of Port Aransas (Tom Brooks, Jim Urban) will provide the GLO and PI Engineering with all available information on the project including:
   i. Digital and/or hard copies of all topographic and bathymetric surveys for the project site
   ii. Detailed information on waterline—specifically cross section and plan view of location of waterline, cross sections (topographic and bathymetry) of area of waterline
   iii. All technical, scientific, engineering, or permitting reports available. Specifically, engineering reports on shoreline projects in or near the project site, such as bulkheads, riprap, and marinas.
4. The GLO (MM) will provide the CPA with a scope of work.
5. The GLO (MM) will clarify communication system regarding direct contact between PI Engineering and the CPA.
6. All parties agreed that PI Engineering will submit bi-weekly technical memorandums to the GLO and Local Sponsor. The GLO and Local Sponsor will review and submit comments to PI Engineering.
7. PI Engineering will expedite the alternatives analysis for the waterline project and make this aspect of the study a priority.

APPENDIX B

Project Review Report
August 8, 2000

Technical Memorandum

Corpus Christi Ship Channel Shoreline Stabilization Project

Project Review

Introduction

This technical memorandum is prepared in accordance with Task 1 of the Scope of Work for the Corpus Christi Ship Channel Shoreline Stabilization Project. The project review is based on the compilation and review of results from previous studies and data available at the time of review preparation. This technical memorandum consists of three sections:

Section No. 1 summarizes the existing conditions of the shoreline along the project site.

Section No. 2 discusses the results of the review and analysis of the permit application prepared by Shiner, Moseley, and Associates, Inc. for the Port of Corpus Christi Authority and the City of Port Aransas.

Section No. 3 summarizes the results of previous shoreline erosion activities along the Port Aransas shoreline.

1. City of Port Aransas Shoreline Along the Corpus Christi Ship Channel Existing Conditions

The existing conditions of the City of Port Aransas shoreline are based on results from site visits conducted by Pacific International Engineering, PLLC (PI Engineering) specialists on July 11, July 19, and July 20, 2000. These results include photographs of the observed shoreline, review and analysis of previous studies, and available design documentation from existing shoreline erosion protection projects. Based on discussions with the Texas General Land Office (GLO) Program Manager, Bill Worsham, and the GLO Project Manager, Mathew Mahoney, it was suggested to present the existing shoreline conditions in the format of a Shoreline Inventory.

A Shoreline Inventory of the Corpus Christi Ship Channel (CCSC) City of Port Aransas shoreline consists of a basemap -aerial photograph of 1995 (Figure 1) and ground-level photographs of the shoreline which were collected during site visits. To display specifics of the City of Port Aransas along the CCSC shoreline, a basemap is divided at cells (grids) S1 through S5 (Figures 2 through 6) (South side of the CCSC, Mustang Island) and N1 through N3 (Figures 7 through 9) (North side of the CCSC, Harbor Island).

Each cell of the inventory is supplemented by a series of ground-level photographs, specifying the typical shoreline configuration and/or existing structures along the shoreline. Information about the type of the structure, year of design, and year of construction, if available is presented in Table 1.1.

Table 1.1 Available Information on Existing Structure

Figure	Item	Description	Date Constructed	Designed by
10	Piper Channel Geotubes	Two, approx. 700 ft long geotubes filled with fine dredged material.	1997	Hartman Consulting Group
15	Rubble	Broken concrete and asphalt, some rebar exposed	Not available	Not available
21	UT Concrete bulkhead	4 ft concrete sheets, approx. 5 ft above water and believed to be embeded 5 ft below mudline.	Not available	Not available
24 A	Boat Launch	Linked concrete cobbles and foundation	199?	Urban Engineers
25	Bulkhead at Park	4 ft concrete sheets topped with concrete cap and sidewalk. Embeded below mudline approx. equal to amount above mudline (5 ft)	1987?	Urban Engineers

The inventory is designed to allow updating and modification of the inventory as new information is available, or changes to the shoreline occurred.

2. Permit Application Review and Analysis

This section of the memorandum contains technical review comments by PI Engineering on the Application to Construct Shoreline Protection Along the Corpus Christi Ship Channel and supporting documents that were prepared by Shiner, Moseley, and Associates, Inc. for the Port of Corpus Christi Authority and the City of Port Aransas. The application seeks to obtain U.S. Army Corps of Engineers (Corps of Engineers) permits for structural measures that are intended to protect the shoreline along Harbor and Mustang Islands from erosion.

The shore protection project is to address erosion of the Harbor Island shoreline along the levee that confines a dredged material disposal site, and Mustang Island shoreline. The total length of shoreline to be protected is approximately 8,000 feet (ft). A location map of the project area is shown in Figure 1 of the Permit Application. Erosion is addressed in four shoreline reaches along the Mustang Island shoreline. The reaches are: (1) shoreline and bottom slope along the water supply line, (2) public fishing pier/park, (3) road to the park site; and (4) eroding shoreline along the CCSC at its confluence with Piper Channel. Wind waves and ship wakes have been identified as contributing factors to erosion at the project area. The planned approach to shore protection is to install structures in the eroding areas.

The structures presented by the permit application are coastal engineering structures that are typically designed for shore protection projects. The initial assessment is that most of the structures described in the drawings have applicability for the physical environment along the CCSC. Performance of these structures, however, depends on the detail of their design and the environmental processes with which they must interact. Hydrodynamics (waves, wakes, and currents), morphology (bottom and shoreline configurations), and littoral conditions (sediment transport) along the CCSC may result in performance of regular coastal engineering structures that is different than expected. The specifics of these environmental processes are explained below:

- Hydrodynamics at the CCSC shoreline account for forces affecting the shore structures that are generated by wind waves (may not be significant); vessel wakes, both bow and stern, (may be significant) pressure field or draw down effects (very significant); current velocities (most likely significant);

- Morphology at the CCSC accounts for effects of the deep channel cut (navigation channel is deeper than the equilibrium depth of the waterway) on the nearby active beach slope; and,

- Littoral conditions describe the potential loss of sand into the channel cut.

While the structures in the permit drawings seemed to be applicable in concept, documentation available for this review did not indicate that the environmental processes listed above had been analyzed sufficiently for the permitting phase of the project. Therefore, the permit application was reviewed from the perspective of determining if the structure is applicable in its proposed setting and to optimize the structure, if recommended, for construction and maintenance costs, project life, and impacts on adjacent areas. The engineering analysis presented below suggests modifications or confirms preliminary design of the structures for the particular shoreline reach in which they are to be installed.

The permit application is lacking important dimensions and details of the structures, which does not allow a comprehensive evaluation of this design. Consequently, our review is limited to the information that is available on drawings and specifications of the permit application.

Shore Erosion Protection along Harbor Island (Page 2, Sheet 2). The project description is placement of a detached breakwater at a short distance from the existing shoreline. This breakwater is constructed of sand-filled geotubes. In select locations, geotube groins are proposed to construct.

Comment 1: The elevation of the toe of geotube is not specified on the drawing, but is assumed to be between elevation +1.0 ft and –2 ft msl (mean sea level), based on information from Sheet 2 cross sections A-A through E-E. Therefore, the geotube crest elevation is between +6 ft and +3 ft msl. Preliminary calculations show that during high tide and in the event of a deep draft vessel passing through the channel at a speed in excess of 8 knots, the geotube will be overtopped by ship-generated waves, where the crest elevation is below +4 ft. Water from the overtopping wave will create a return flow in the gaps between the breakwater segments that might undermine the geotube foundation itself. Scour protection, either with an apron of geotextile or coarse stable material in the gaps, could be designed to prevent undermining the geotubes. At high tide, waves could overtop the geotube to such an extent that the toe of the bluff at the backshore can be eroded. Crest elevation will require more analysis than was evident from the drawings.

Comment 2. The connection of the geotube with the groin (Sheet 2, Section F-F) will allow water to flow through the gap. Our estimate shows that water velocity could exceed 6 ft per second in the gap. Such flow speed is expected to significantly scour the beach under the geotube and the groin, which could result in the failure of the structures. The Piper Channel geotube project demonstrated that a scour hole for the type of connection shown in the drawing could reach 10-15 ft in a short period of time.

Miscellaneous Comments:

In the table "Typical Dimensions (Sheet 2) Z is denoted as the length of the geotube groin. In the typical cross section F-F, Z includes a geotube groin plus half the width of shore-parallel geotube. For clarity purposes, the dimension should refer only to the length of the geotube groin.

Note 3 specifies that there will be gaps in the breakwater. No information on gap size and locations is provided, but is crucial for estimating scour and scour protection.

It is not clear how the information in Note 6 (Sheet 2) is related to the project and drawing.

Conclusion: Detached breakwater and groins constructed from geotubes may not be applicable for the Harbor Island shoreline conditions. A more detailed engineering analysis will determine the applicability of these structures.

Shoreline along the Waterline Project, Page 2, Sheet 4, bulkhead or sloped revetment is positioned along the approximate location of the former shoreline. The area behind the bulkhead (or revetment) will be filled up to the grade of the remaining upland.

Comment 1. Typical Profile (Sheet 4) shows a particular ground profile at the toe of the bulkhead. Experience has shown that for certain bulkhead toe elevations and material types at the toe, a scour hole could form in front of the structure. Toe protection would need to be designed to prevent scour. No indication of toe protection is found in the permit application.

Comment 2. Bulkhead or revetment is aligned parallel to the shoreline at approximately 150 ft seaward from the existing shoreline. An analysis of historical data and construction drawings (Urban Engineering 1988 and Reagan & McGaugham Engineers, 1968) suggests that the pipeline is submerged under the bottom surface less than at 2 ft at this location. Construction of the bulkhead will not stop the ongoing scouring process in shallow water in front of the bulkhead. Consequently, after constructing the bulkhead or revetment along the proposed alignment, bottom lowering will continue. Considering that the thickness of sand above the pipe at the intersection with the bulkhead is only two ft or less, the waterline pipeline may be exposed again relatively soon.

Comment 3. Considering the shallow depth of pipeline under the ground line, it is not clear how the bulkhead can be placed at the proposed location. The pipeline could be damaged by the placement of the bulkhead unless the bulkhead could bridge over the pipe. A standard bulkhead at the pipeline crossing might not be feasible.

Conclusion: A bulkhead or revetment at the proposed alignment may be a short-term solution of protecting the pipeline. An engineering analysis will be completed to optimize the alignment of the bulkhead, or to develop a more feasible alternative for this part of the shoreline.

Shoreline along the access road and fishing pier's east flank, (Page 2, Sheets 5 and 8). Revetment, bulkhead, or riprap on the waterside of the existing rubble and fill over the rubble.

Comments 1. Neither sheets 5 or 8 cross sections show how the existing rubble is integrated into the new shore protection. The type of shoreline protection is not specified.

Comment 2. Because of the occasional high energy level of vessel-generated waves in this environment, it is likely that the size of riprap or revetment necessary to remain stable will create unsafe conditions for public access to the shoreline.

Comment 3. A critical element of the project is an access road protection structure. This part of the project may be the most expensive, based on the plan view on Sheet 5. However, no proposed cross section (even typical) is developed and presented in the permit application for this shoreline reach.

Comment 4. It is not clear what a dashed line on the plan view (Sheet 5) means. It appears that the same dash line on Sheet 6 denotes a vertical bulkhead. However, no discussion on bulkhead structure is found for the access road and east flank of the fishing pier shoreline protection in the permit application. Section A – A is called out on the plan, but the section is not shown. Cross section A – A is shown on Sheet 6, but refers to a different location of a section A – A.

Conclusion: This part of the shore protection has not been developed to the level of an engineered structure. If a bulkhead is intended, its design features should be shown on Sheet 5. Riprap and revetment may not be conducive to safe public access to the shoreline.

Shoreline of the public park west of the fishing pier about 1000 ft long. (Page 2, Sheet 6) Construct bulkhead or revetment, or combination of the two, on the water side of the rubble and fill the area over the rubble and behind the bulkhead.

Comment 1. Because of occasions of higher wave energy in this environment, it is likely that the riprap may not be safe for public access.

Conclusion: Bulkhead structure shown on Sheet 6, cross section A - A, may be accepted as a concept for this part of the shoreline. Revetment or riprap structures for this part of the shoreline has not been developed to the level of engineered structures.

***Shoreline between the park and Piper Channel approximately 8,000 ft long*.** (Sheet 2 and Sheet 7) A combination of bulkhead, geotubes, and revetment, possibly supplemented by offshore breakwater or groin, with 100-ft-wide public access easement.

Conclusion: The shoreline protection at this location has not been developed to a level of engineered structures. Specific shore protection designs corresponding to particular locations along the shore must be developed.

Section Summary

- The structures shown in the Permit Application are typical coastal engineering structures frequently designed for various shore protection projects. However, considering particular hydrodynamic, morphological, and littoral transport conditions along the CCSC, performance of the structures may be different than expected, and supporting engineering analysis is not apparent from documentation in the application.

- The detached breakwater and groins described for shore protection along Harbor Island may not be applicable to this area.

- The bulkhead or revetment described for the water line crossing the project may be a short-term solution for protecting the pipeline. Engineering analysis should be completed to optimize the alignment of the bulkhead (if this alternative is preferred), or develop a more favorable alternative for this part of the shoreline.

- Shore protection along the public park west of the fishing pier, the access road, and the fishing pier's east flank has not been developed to the level of an engineered structure, and therefore detailed comments cannot be made on the design.

- An engineering analysis, including development of the alternatives and selection of a preferred alternative, should be completed for all parts of the shoreline along Harbor and Mustang Islands in order to obtain a shore protection system that is durable in the long-term, minimizes unwanted environmental impacts, and is economical.

- Permit drawings require certain information that is missing on Sheets 1 – 9. Single quantities of excavation and fill are needed, not ranges of volumes. "Slightly" is not specific enough for use in this permit application. There should be no confusion that no wetlands will be filled if that is the case. Drawings need north arrows, elevation datums, Mean High Water elevation on cross sections, and Mean High Water line on all plan drawings. Rock sizes should be stated. Geotube construction details should be provided, such as material type, footprint, and equipment to be used in construction. All structure dimensions and elevations should be called out.

- A pre-application meeting should be held with permitting agencies to work out concerns, clarify project description, and to determine the level of detail to provide with the application. It is apparent that this has not yet happened.

3. Port Aransas Shoreline Erosion Previous Studies Review

We have compiled, reviewed, and analyzed available engineering studies associated with the shoreline erosion along the Port Aransas shoreline. Such studies were conducted in association with the deepening of the navigation channel (Corps 1994) as well as for various other purposes directly or non-directly related to the shoreline processes (Texas A&M, 1976, Kraus 1995, and Hartman, 1996)

The review of previous studies shows that the erosion along the CCSC shoreline for both, Harbor and Mustang Islands, has been observed for the last several decades. The estimates for the rate of erosion by the previous studies do not conflict and provide a relatively comprehensive evaluation of the shoreline recession over a long period of time. For example: the average rate of erosion is estimated at 12 ft per year (City of Port Aransas, 2000).). However, the rate of shoreline erosion is not constant and varies in time and distance along the channel. On average, over the last 30 years the rate of erosion was estimated at approximately 17 ft per year. The rate of erosion before the channel deepening in 1975 was estimated at less than 12 ft per year. The rate of erosion has increased over the last two decades and was measured at some locations at a rate of more than 40 ft per year (Hartman Consulting Group, 1996).

Previous studies concerning the CCSC shoreline erosion had discussed potential contributing factors of erosion. Conclusions from these studies do not conflict, but rather compliment each other. The causes of shoreline erosion as identified by previous studies are summarized below.

- Waves generated by the ship traffic (Corps of Engineers, 1994, Hartman, 1996);
- Drawdown effect (S&M Permit Application 2000, Hartman, 1996)
- Wind waves (Hartman 1996)
- Channel morphology and maintenance dredging (Hartman 1996).

Another potential cause of erosion was identified by the PI Engineering team during a site visit on July11, 2000. This cause includes current velocities generated in the CCSC by tidal fluctuation. It was later confirmed based on discussions with pilots of the Corpus Christi and Port Aransas Pilot Association, that these velocities may reach in excess of 6 ft per second and are capable of scour in addition to transporting a significant amount of sediment.

Field data available from previous studies regarding wave, currents, sediment transport, and bathymetry along the CCSC are limited. The sources of these data were found in studies conducted by Kraus (1995) and Texas A&M (1976). Additionally, the source of current velocity data was found in the Conrad Blucher Institute home page.

Data collected in 1975 by Texas A&M includes wave data at the Barge Slip Harbor area and includes records of long period waves generated by a deep draft vessel. The detailed data on the deep draft vessel that generated these long period waves is also available from this study. Preliminary analysis of these data indicates a potential influence on the wave record from the harbor resonance. If this is proved, these data may not be applicable for the shoreline erosion study.

Data collected by Kraus (Kraus 1995) are limited to the topographic and bathymetric survey and current data collection in the Piper Channel. The area of survey covers approximately 3,000 ft of shoreline in the vicinity of Piper Channel. These data can be partially applied to the analysis of the alternatives for the shoreline erosion at the Piper Channel shoreline.

The Texas Water Development Board (TWDB) also has ADCP data for the channel reach between the ferry terminal and Piper Channel. This data may be available only in a 2 to 3 week period. The schedule of these data may prevent their use for the project. Additionally, current meter data available through the UT Marine Science Institute is being analyzed and will be used in determining currents in and along the CCSC.

Section Summary

- Shoreline erosion along the CCSC shoreline is a long-term process with rates of erosion that vary in time and space (along the CCSC length);
- Previous studies have determined qualitatively the range of causes of erosion. No quantifying estimates for the potential causes had been developed.
- A field data collection program conducted under the current study will fill the gap in the vessel, wave, vessel wakes, bathymetric, and topographic survey data sets that are required for the alternatives analysis.

4. References

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