



# Preliminary Design Memorandum

Salt Flats Levee System, Phase 2, Task Order 5

**City Project Number:** E12070

**HDR Project Number:** 2220658

**City Funding Source:** Type A Corporation, Seawall  
Dedicated Sales Tax

*Corpus Christi, TX*  
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## PRELIMINARY

THIS DOCUMENT IS RELEASED FOR THE PURPOSE OF INTERIM REVIEW AND IS NOT INTENDED TO BE USED FOR CONSTRUCTION, BIDDING OR PERMIT PURPOSES.

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## TABLE OF CONTENTS

1. Introduction .....	1
AUTHORIZATION .....	1
PURPOSE & SCOPE .....	1
2. Site Investigation .....	4
Topographic and Hydrographic Surveys .....	4
Field Visits .....	4
3. Geotechnical Considerations .....	6
Field Investigation .....	6
Settlement .....	7
4. Wave Analysis .....	9
Still Water Elevation .....	9
Relative Sea Level Rise .....	9
Total Still Water Level .....	9
Wave Numerical Model .....	10
Calculation of Wave Loads on Proposed Structure .....	13
Calculation of Wave Overtopping .....	13
Results .....	14
5. Wall Considerations .....	15
Alignment .....	15
Impacted Utilities .....	15
Flap-Gate Box .....	15
Gas / Water / Fiber Optic .....	15
Wall Sections .....	15
Free-Standing Floodwall .....	16
Inverted-Tee Retaining Wall .....	18
Sheet Pile Retaining Wall .....	19
Slope Stability .....	20
Wall Size and Length Cases .....	22
Factors of Safety .....	22
6. USACE Permitting .....	24
7. Opinion of Probable Project Cost .....	25
Barge Impact Costs .....	25
8. Report Limitations .....	27
9. Items Pending for Final Design .....	28

## TABLES

Table 1 - Design Soil Profile.....	7
Table 2 - Still Water Elevations considered for Design.....	9
Table 3 - USACE Projected Relative Sea-Level Change at NOAA Station 8774770 Rockport, TX .....	9
Table 4 - Total Still Water Elevations Considered for Wave Analysis .....	10
Table 5 - Spectral Wave Model Results.....	12
Table 6 - Summary of Wave Load and Overtopping Calculations.....	14
Table 7 - HSDRRS Design Guide Impact Loads .....	17
Table 8 - Slope Stability Analysis Summary .....	22
Table 9 - Wall Design Cases.....	22
Table 10 - Minimum Safety Factors for Determining Penetration.....	23
Table 11 - Allowable Stresses for Design of Sheet Pile Walls.....	23
Table 12 - OPPC for Sheet Pile Retaining Wall.....	25
Table 13 - Barge Impact Comparison.....	25

## FIGURES

Figure 1 - Portion of City of Corpus Christi Flood Protection System.....	2
Figure 2 - Project Limits .....	2
Figure 3 - Photo of Flap Gate.....	4
Figure 4 - Museum Wall Boring Locations .....	6
Figure 5 - Data Extraction Points for Wave Model.....	11
Figure 6 – Example of Wave Modeling Results (Case 5).....	13
Figure 7 - Free-standing Inverted-Tee Floodwall Section.....	16
Figure 8 - Boat/Barge Impact Map New Orleans Area (Source: HSDRRS).....	17
Figure 9 - Inverted-Tee Retaining Wall Section.....	18
Figure 10 - Sheet Pile Retaining Wall Section .....	20
Figure 11 - Slope Stability Wall Section Diagrams, Wall near Slope.....	21
Figure 12 - Slope Stability Section Diagrams, Wall with Bench.....	21

## **APPENDICES**

Appendix A: Museum Wall Survey

Appendix B: Geotechnical Report

Appendix C: Existing Utilities

Appendix D: Opinion of Probable Cost (OPPC)

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# 1. INTRODUCTION

## AUTHORIZATION

The work outlined in this study was authorized by representatives of the City of Corpus Christi, Texas (City). The work performed herein is in accordance with HDR Engineering, Inc. (HDR) Task Order No. 5 and authorized by Mr. Jeff Edmonds, PE on May 3, 2016, with Mr. Daniel Deng, P.E., serving as the City's Project Manager. This work is part of the overall assessment for the Salt Flats Levee System, City Project Number E12070.

## PURPOSE & SCOPE

As part of the development of the Preliminary Flood Insurance Rate Maps (FIRMs) for Nueces County, in March 2012 the Federal Emergency Management Agency (FEMA) requested documentation from the City of Corpus Christi showing compliance with the requirements in 44 CFR 65.10 for the Salt Flats Levee and Museum Floodwall. In response, the City commissioned a study by Urban Engineering which produced a certification guidance report in December 2012. A Capital Improvement Plan (CIP) project was included in Group 3J – Public Health and Safety in the CIP 2012-13 for the Salt Flats Levee Certification – Phase 2 to fund the final design and construction of the improvements. Phase 2A was an initial evaluation of the existing data available for the Salt Flats Levee and Museum Floodwall, to identify and develop a plan to correct deficiencies to allow certification of full compliance with 44 CFR 65.10 as a Sound Reach.

In July 2013, FEMA finalized a new approach for Levee Analysis and Mapping Procedures (LAMP) for Non-Accredited Levee Systems, and launched the Nueces LAMP Pilot Study to determine how the protected area will be shown on the effective FIRM maps. The LAMP approach breaks the flood protection system down into reaches, and offers five potential procedures for each reach, including Sound Reach (full compliance with 44 CFR 65.10) and Freeboard Deficient Reach (does not meet freeboard requirements, but complies with the rest of 44 CFR 65.10). Because both the Sound Reach and Freeboard Deficient Reach procedures result in similar maps for downtown protected area, in August 2015 the City determined that it would be more cost effective to address structural and/or operational deficiencies but not raise the levee (or any of the other flood protection components) to provide the required freeboard.

The existing I-wall structure at the Museum of Science and History, which is shown in Figure 1 and referred to as Museum Floodwall herein, is of unknown structural integrity, and was constructed at a time when design standards for flood protection structures were less than they are today. Failure or overtopping of this wall could result in flooding of landward areas. The City has decided to replace the Museum Floodwall with a new flood protection structure that will extend from the Promenade under the Harbor Bridge (to the west) to the U.S. Army Corps of Engineers (USACE) bulkhead (to the east) (see Figure 2).



Figure 1 - Portion of City of Corpus Christi Flood Protection System



Figure 2 - Project Limits

HDR was scoped to conduct a geotechnical investigation and site survey, perform wave analysis specific to the project location, identify potential USACE regulatory requirements, and perform a preliminary design of the proposed wall. This report details the investigation, analysis and design performed to generate the 30% design drawings and Opinion of Probable Project Cost (OPPC) for replacement of the Museum Floodwall.

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## 2. SITE INVESTIGATION

### TOPOGRAPHIC AND HYDROGRAPHIC SURVEYS

A topographic and hydrographic survey of the project site was performed by Naismith Marine Services in June 2016. The area of land surveyed was between the eastern side of the parking lot connected to Mesquite Street and the fence separating the USACE / City of Corpus Christi properties. The limits of the survey in the north-south direction were the approximate centerline of the Corpus Christi Ship Channel and the concrete floodwall adjacent to the Corpus Christi Museum of Science and History.

The vertical datum for the hydrographic and topographic survey was the North American Vertical Datum of 1988 (NAVD '88). The survey results can be found in Appendix A.

### FIELD VISITS

Field visits were performed in order to identify likely obstructions or obstacles to be considered during the design and construction of the new flood protection structure. One observation includes an obstructed flap-gate box and has been noted on the 30% drawings.

HDR was unable to enter the flap-gate box at the site because it would require special “confined space entry” safety provisions, but photographs were taken from the surface. Figure 3 contains one photo of the flap-gate within the concrete box. The photo shows the flap-gate has visible rust, section loss due to corrosion, and is in poor condition.



**Figure 3 - Photo of Flap Gate**

Also, there appear to be rocks at the bottom of the flap-gate box that could interfere with the operation of the flap-gate in the event of a storm surge. HDR recommends that the City remove the rocks from the bottom of the flap-gate box to ensure the flap-gate is operational as intended.

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### 3. GEOTECHNICAL CONSIDERATIONS

A geotechnical investigation was conducted at the site and analyses were performed. The following sections describe the field investigation.

#### FIELD INVESTIGATION

Two soil borings were performed at the location of the proposed replacement structure, as shown in Figure 4. The borings were performed by Rock Engineering and Testing Laboratory, Inc. under contract with the City. The borings were advanced to a depth of 60 feet below the existing ground surface. The details from the soil testing program can be found in Appendix B.



Figure 4 - Museum Wall Boring Locations

Based upon the laboratory testing of the soils analyzed in soil profiles B1 and B2, a single soil profile was developed for design. Upon evaluation of the soils in Borings B1 and B2, Boring B1 was found to generally consist of soils that have friction angles and/or undrained shear strengths that are considered to be less advantageous for the design of a wall than Boring B2. Therefore, as a conservative measure the soil profile from Boring B1 was applied for preliminary design of the wall; this profile is shown in Table 1.

**Table 1 - Design Soil Profile**

Top Elevation	Description	Unit Weight (pcf)	Undrained Shear Strength $C_u$ (psf)	Drained Parameters	
				$\Phi'$ (deg)	$c'$ (psf)
12.5	PROPOSED FILL*	125	0	30	-
6.0	Very Stiff Clay	120	1800	30	150
0.0	Medium Dense Sand	117.4	-	30	0
-3.0	Very Soft Lean Clay	108.4	250	26	50
-8.0	Very loose Clayey to Silty Sand	110.4	0	28	0
-15.0	Soft Fat Clay	117.4	500	27	100
-44.0	Dense Silty Sand	119.4	-	35	0
-50.0	Stiff Fat Clay	122.4	1800	27	200

\*Note: The parameters for the PROPOSED FILL are assumed and should be verified during final design.

## SETTLEMENT

There are several layers of soft and loose material in the soil profile along the alignment of the proposed wall. The addition of fill along the landward side of the wall is likely to cause settlement.

As previously discussed, there were two borings performed along proposed the alignment of the replacement wall (hereafter referred to as the "Museum Wall") and the encountered soil conditions contained variable soil properties at the two exploratory borings. This suggests the material properties and stratification are variable across the fill area and the settlements should be expected to be variable as well. The lateral extent of the fill could vary between 80 feet near the mid-point of the wall alignment and 2-10 feet near the tie-ins. Therefore, it is likely that little to no settlement will be experienced at the ends of the wall and is likely the maximum settlement will be experienced near the midpoint.

The soil borings contained several layers of sand and clay layers. Sand layers will experience more immediate settlement and clay layers will experience settlement over time. A summary of the immediate and time dependent settlement is provided below:

### Immediate Settlement:

- The maximum amount of immediate settlement is expected to be between 1 and 1.5 inches.
- The minimum amount of immediate settlement will likely approach zero (at the ends of the wall).
- The immediate settlement will occur during the fill and compaction efforts that take place during construction. The immediate settlement will have taken place before the wall is built so it will not impact elevations of the completed wall.

### Time Dependent Settlement (Consolidation)

- The estimated maximum anticipated amount of consolidation settlement is approximately 12-16 inches.
- The minimum amount of consolidation settlement will approach zero at the tie-ins.

- Based upon experience, the settlement should be expected to be highly variable across the fill area, though generally ranging from 6 to 12 inches.
- It is recommended that construction of any flatwork or pavement behind the wall be delayed until most of the consolidation settlement has occurred. It would be reasonable to expect a majority of the consolidation settlement to occur within the first 2 or 3 years, but this will be verified in final design.
- If there is no pavement or flatwork behind the wall, there is essentially no issue associated with the consolidation settlement other than possibly some minor grading to fill in settlement depressions and/or maintain drainage.
- The magnitude of the expected consolidation settlement is rather significant and will likely impose a certain amount of down drag on the wall. There are unlikely to be issues with plunging or bearing failure of the sheets for the anticipated pile depths, but some vertical movement is possible. Even if small, the vertical movements could induce some strains within the concrete exposed portion of the wall. This is a consideration for final design.
- The computed maximum immediate settlement is rather modest and therefore unlikely to introduce significant downdrag forces on the wall.

#### **Pre-Loading:**

One way to reduce the impact of the settlement on the paving, retaining wall and other proposed features on the site is to pre-load the soil. Pre-loading the soil consists of adding the proposed compacted backfill to the site prior to constructing the proposed wall. The pre-loaded area would be given time to allow settlement to occur prior to constructing the wall.

After the soil has had time to settle, the proposed wall and site improvements could be constructed. This would mitigate some of the issues associated with settlement of the backfill. The time required for settlement to occur has not been determined and will need to be verified in final design.

## 4. WAVE ANALYSIS

Wave load and overtopping calculations were performed for the proposed flood protection structure. These analyses were supported through development of a MIKE21 Spectral Wave (SW) numerical model. This section documents the analyses performed for storm surge water levels and waves, including both wave loading and overtopping.

### STILL WATER ELEVATION

Storm surge still water elevations (SWE) as a function of return period were obtained from the Federal Emergency Agency (FEMA) Region 6 Flood Insurance Study (FIS): Coastal Counties, Texas report dated 15 November 2011, as summarized in Table 2. These elevations represent storm surge without wave effects.

**Table 2 - Still Water Elevations considered for Design**

Return Period	Still Water Elevation (ft., NAVD)
10	+5.0
50	+6.9
100	+8.4

### RELATIVE SEA LEVEL RISE

Values of Relative Sea Level Rise (RSLR) were obtained from the Sea-Level Change Curve Calculator developed by USACE. The RSLR values were calculated at NOAA Station 8774770 Rockport, TX, using 2016 as the present year. A summary of the RSLR values considered for design is listed in Table 3. These values were considered in selecting the total still water level (SWE + RSLR) for evaluation of potential wave loads and wave overtopping at the proposed wall.

**Table 3 - USACE Projected Relative Sea-Level Change at NOAA Station 8774770 Rockport, TX**

Year	Relative Sea Level Rise (ft.)
2016 (0-years)	+0.0
2066 (50-years)	+1.3
2116 (100-years)	+3.0

### TOTAL STILL WATER LEVEL

To evaluate potential extreme storm and/or long-term design cases, relative sea level rise was added to the SWE to calculate a total still water elevation. For the 10-year and 50-year return period water level, no allowance for sea level rise was included. For the 100 year water level, varying levels of RSLR from 0 to



the 100 year projection were considered. The total still water levels evaluated for wave analysis are presented in Table 4.

**Table 4 - Total Still Water Elevations Considered for Wave Analysis**

Return Period	Relative Sea Level Rise (ft.)	Total Still Water Level (ft., NAVD)
10	+0.0	+5.0
50	+0.0	+6.9
100	+0.0	+8.4
100	+1.3 (50 years)	+9.7
100	+3.0 (100 years)	+11.4

## WAVE NUMERICAL MODEL

A MIKE21 SW wave model was developed to simulate wave conditions at the proposed wall. For the selected design case, the 100-year water level was modeled with a sustained windspeed of 80 mph in accordance with the Federal Emergency Management Agency’s (FEMA’s) Flood Insurance Study (FIS) procedures. This windspeed matched the ASCE7-10 100-year 3-sec gust wind speed which was converted to a 20 minute sustained wind speed of 80 mph. The direction of the wind forcing was varied in 10 degree increments. After the controlling direction of 70 degrees clockwise from north was determined, the 80 mph sustained wind speed was also modeled at the lower water levels listed in Table 3; these lower water levels were evaluated for comparative purposes. Model output (wave height and period) were extracted at two locations near the project site (see Figure 5). Point 1 (Pt 1) represents wave conditions at the toe of the proposed wall and Point 2 (Pt 2) represents wave conditions within the Corpus Christi Ship Channel. Table 5 summarizes the results from the modeling effort. An example of the wave modeling results is shown in Figure 6.



**Figure 5 - Data Extraction Points for Wave Model**

**Table 5 - Spectral Wave Model Results**

Case	Return Period (years)	Total Still Water Level, ft (NAVD)	Parameter	Point 1	Point 2
1	10	+5.0	Significant Wave Height (ft)	1.0	5.9
			Max Wave Height (ft)	0.6	11.8
			Peak Period (sec)	5.4	4.4
			Mean Direction (deg)	50.4	86.7
2	50	+6.9	Significant Wave Height (ft)	3.5	6.1
			Max Wave Height (ft)	2.0	12.3
			Peak Period (sec)	4.8	4.6
			Mean Direction (deg)	49.6	86.5
3	100	+8.4	Significant Wave Height (ft)	4.0	6.3
			Max Wave Height (ft)	3.0	12.6
			Peak Period (sec)	5.0	4.9
			Mean Direction (deg)	53.0	86.4
4	100	+9.7	Significant Wave Height (ft)	4.5	6.4
			Max Wave Height (ft)	4.0	12.9
			Peak Period (sec)	5.1	5.0
			Mean Direction (deg)	55.7	86.2
5	100	+11.4	Significant Wave Height (ft)	4.9	6.6
			Max Wave Height (ft)	5.1	13.4
			Peak Period (sec)	5.3	5.1
			Mean Direction (deg)	58.9	86.3

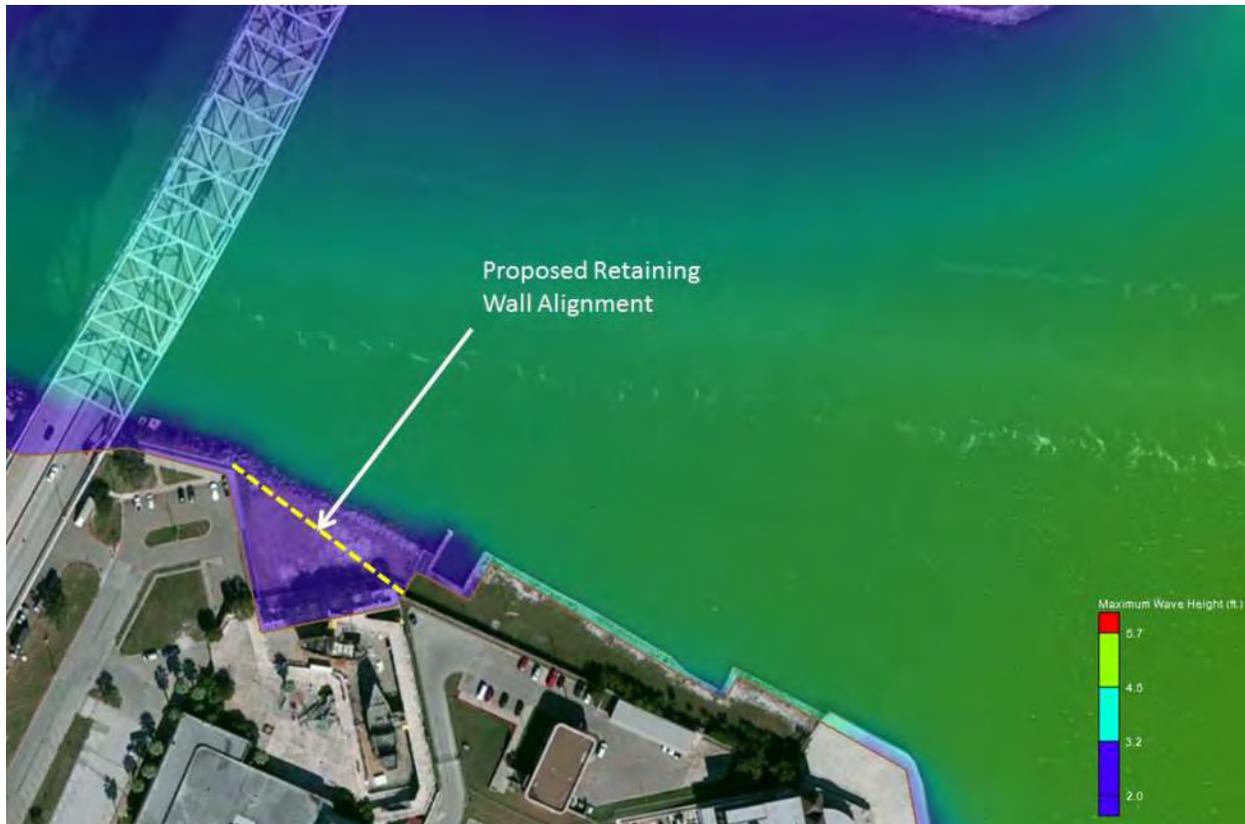


Figure 6 – Example of Wave Modeling Results (Case 5)

## CALCULATION OF WAVE LOADS ON PROPOSED STRUCTURE

The proposed wall will tie-in at the same elevation (approximately +12.7 ft NAVD) as the adjacent flood protection structures and is expected to be overtopped by waves during a 100-year design event. Based on a survey conducted by Naismith Marine Services in June 2016, the existing ground elevation seaward of the proposed retaining wall is approximately +5.1 ft NAVD, leaving approximately 7.6 feet of wall exposed to potential wave impacts. The calculation of wave loads on the proposed structure was performed following the methodology developed by Goda and as described in the USACE Coastal Engineering Manual.

## CALCULATION OF WAVE OVERTOPPING

Average wave overtopping discharge was calculated using the formula developed by Franco and Franco (1999) as described in the USACE Coastal Engineering Manual.



## RESULTS

Table 6 summarizes the results of the wave analysis. For structural analysis, Case 5, which yielded the greatest wave load, was applied. To assess the need for scour protection along the landward side of the wall, the wave overtopping results were applied. For levels of overtopping up to Case 4 (100 year SWE plus 50 years of SLR), the area landward of the wall can be protected from erosion through construction of a splash pad along the crest. For Case 5, which represents a very conservative wave overtopping scenario (100 year SWE plus 100 years of SLR), damage would be expected landward of the wall. Therefore, if the desired service life of the wall is determined to be 100 years, the height of the wall (and adjacent components of the overall flood protection system) should be given consideration for a higher crest at some point in the future. For the present design, the level of wave overtopping expected for Cases 1 through 4 was determined acceptable assuming a splash pad is included.

**Table 6 - Summary of Wave Load and Overtopping Calculations**

Case	SWE (ft., NAVD)	Wave Force (lb/ft)	Resultant Elevation (ft NAVD)	Overtopping (ft <sup>3</sup> /s per ft)	
				Discharge (ft <sup>3</sup> /s per ft)	Potential for Erosion and/or Damage Landward of Wall
1	5.0	3,835	8.5	0.00	No damage expected
2	6.9	4,607	8.6	0.03	No damage expected
3	8.4	(not calculated)	N/A	0.3	No damage expected
4	9.7	(not calculated)	N/A	1.3	Expect erosion if area behind crest not protected
5	11.4	5,489	8.9	6.4	Expect damage

## 5. WALL CONSIDERATIONS

This section details the design considerations for the proposed Museum Wall.

### ALIGNMENT

The alignment for the proposed flood protection structure is shown in Figure 4. This alignment would allow the existing floodwall to be abandoned in place, and provide additional public use space between the new structure and the existing wall.

### IMPACTED UTILITIES

There are existing utilities that are in the area of the proposed wall including stormwater, gas, and water. This section describes the impacts the project may have on these utilities.

#### FLAP-GATE BOX

At the site there is a concrete flap-gate box that has a 24" diameter concrete pipe inlet connecting to the downtown Corpus Christi stormwater system. On the opposite side, there is a 24" diameter concrete pipe for the effluent and it is connected to an outfall. The purpose of the box is to house a flap-gate that prevents seawater from entering into the storm water system when there is not storm water being discharged from the line.

The location of the flap-gate box is near station 0+70 of the Museum Wall and is located within alignment of the wall. Additionally, the top of the structure is around the existing ground elevation (approximately elevation 5.00 – 6.00 ft NAVD '88) and is lower than the proposed top of wall elevation (approximately +12.00 ft. NAVD'88). Therefore, the structure will need to be demolished and relocated with the alignment of the existing wall. It may be possible to modify the alignment of the wall in the final design phase of the project.

#### GAS / WATER / FIBER OPTIC

There are a number of gas and water lines that cross the alignment of the wall and cross the Corpus Christi Ship Channel near Station 0+30. The record documents used to determine the elevations for the utilities placed in the Gas Main Crossing project are located in Appendix C.

In order for the utilities to pass underneath the Corpus Christi Ship Channel, the utilities are required to be at an elevation 30 ft under the future depth of the Corpus Christi Channel (-60.0' MLT = -61.6' NAVD '88). Therefore, the highest elevation the gas and fiber optic lines may be is elevation -90.0' MLT (-91.6' NAVD '88). The deepest tip elevation of the sheet piles is approximately -32.5 ft NAVD '88. The existing utilities are significantly below the anticipated tip elevation of the proposed wall; in addition, the utilities are not significantly close (horizontally) to the proposed wall.

### WALL SECTIONS

Several wall sections were evaluated as replacements for the floodwall. The various sections included a free-standing floodwall, an inverted-tee retaining wall, and a sheet pile retaining wall section. Descriptions of the evaluated wall sections are provided on the following pages.

### FREE-STANDING FLOODWALL

The first wall section that was evaluated is a free-standing floodwall as is shown in Figure 7. The free-standing floodwall would consist of a concrete inverted-tee wall with concrete splash pads on either side of the wall. The purpose of the splash pad would be to prevent erosion of the soil adjacent to the floodwall due to wave reflection and overtopping.

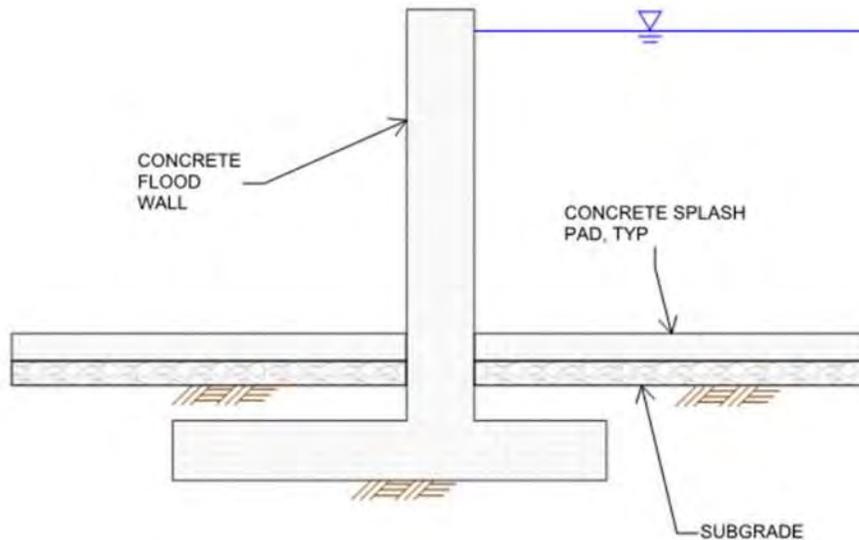


Figure 7 - Free-standing Inverted-Tee Floodwall Section

### BENEFITS

Among the primary benefits of the free-standing inverted-tee flood wall is the shallow embedment of the wall and lack of impact to the surrounding area. Because the embedment is shallow, the area impacted by the construction of the wall would not be significant. Additionally, there would be little to no impact to the existing features on the site.

### CHALLENGES

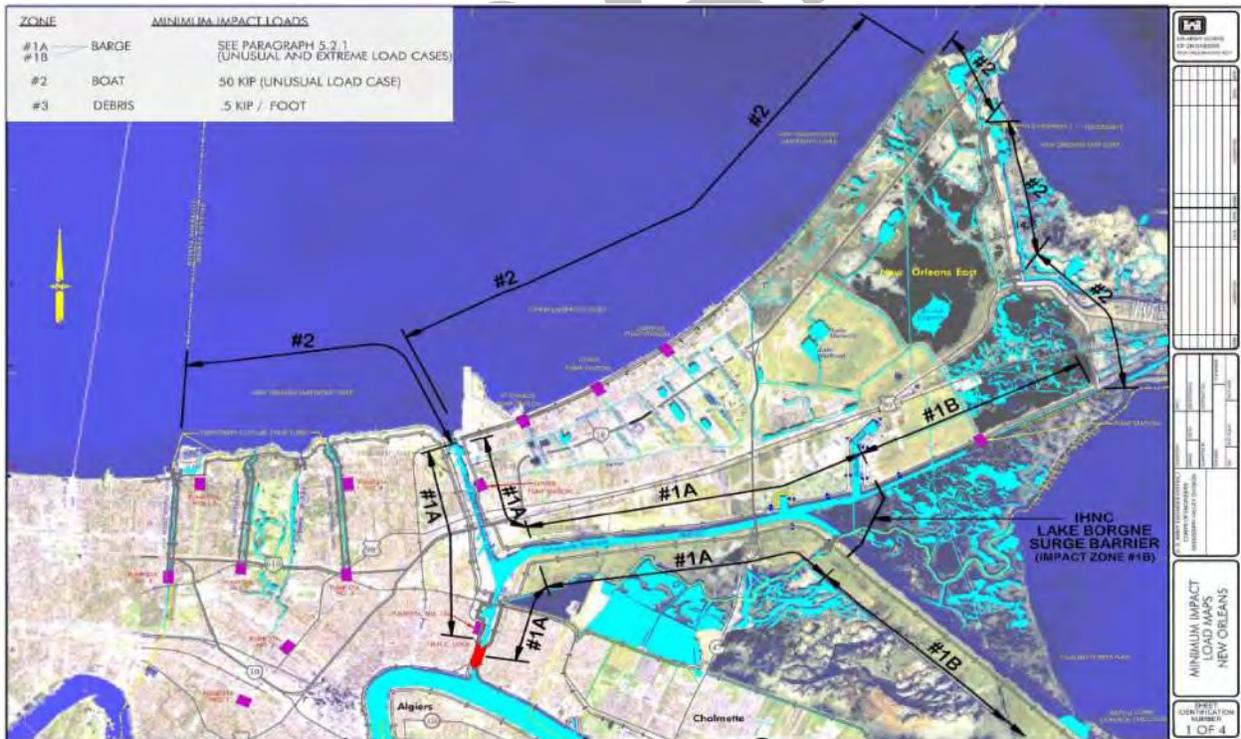
One of the primary challenges associated with a free-standing inverted-tee floodwall is designing for debris, small craft, and barge impact loads. Following Hurricane Katrina, Congress authorized and funded the construction of the 100-year level risk reduction system, known as the Hurricane and Storm Damage Risk Reduction System (HSDRRS). Design guidelines were published for the HSDRRS, which contain requirements for the design of floodwalls, levees, mechanical, etc. aspects of the flood protection system. The HSDRRS design guidelines only apply to the HSDRRS located in New Orleans, but they contain useful guidance on design aspects such as impact loads, factors of safety, and slope stability analysis.

The HSDRRS design guidelines outline the design loads and combinations for each section of the HSDRRS. Because the floodwall would be freeboard deficient, the design water level is nearly at the top of the wall and, similarly, all impact loads would be expected to act at the top of the wall.

The HSDRRS breaks the project into several zones, which are shown on page 5-47 of the HSDRRS. Each of the various zones has specific impact requirements. Recommendations for impact loads in the HSDRRS design guidelines are outlined in Table 7. The Zones mentioned in the table are shown in Figure 8.

Table 7 - HSDRRS Design Guide Impact Loads

Location	Impact Load
Zone 1A	<i>Southeastern Louisiana, protected waterways</i>
	200 kips at the top of the wall (Usual Case)
	400 kips at the top of the wall (Extreme Case)
Zone 1B	<i>Southeastern Louisiana, waterways directly exposed to tidal surge</i>
	225 kips at the top of the wall (Usual Case)
	450 kips at the top of the wall (Extreme Case)
Zone 2	Small Craft Loads – 50 kips applied at the top of the wall
Zone 3	Small Debris Loads – 0.5 kips / LF of wall



HDR contacted Ray Harris (Harbormaster) with the Port of Corpus Christi Authority (Port) to assess the likelihood of barges in the Port of Corpus Christi Inner Harbor during a hurricane. The Port informed HDR that their procedure for hurricane preparedness was to request that all vessels leave the Port prior to landfall of a hurricane, but they could not force the vessels to leave their port. Therefore, there would likely be, and has been in the past, barges in the Inner Harbor during hurricane events.

The proposed wall will be located in a zone that would likely be categorized in Zone 1B because it is in a location where there is barge traffic and it is directly exposed to storm surges. Therefore, the flood wall would need to be designed for 225 kips in the Usual Case and 450 kips in the Extreme Case. Because of the very large magnitude of these loads, it would not be cost feasible to design a free-standing wall to withstand them.

Additionally, if a barge impact did occur on a free-standing wall during a hurricane and the impact load was not included in the design, the failure could be catastrophic and cause a breach in the flood protection system. There is less redundancy to a free-standing inverted tee floodwall as compared to a coastal structure (i.e. a retaining wall, seawall, etc.).

### INVERTED-TEE RETAINING WALL

The next wall section that was evaluated is an inverted-tee retaining wall, which is shown in Figure 9. The inverted-tee retaining wall would consist of a concrete inverted-tee wall that has retaining soil on the landward side of the wall and, in the event of a hurricane, water on the opposite side. Scour around the immediate vicinity of the floodwall would be mitigated using concrete splash pads on either side of the wall.

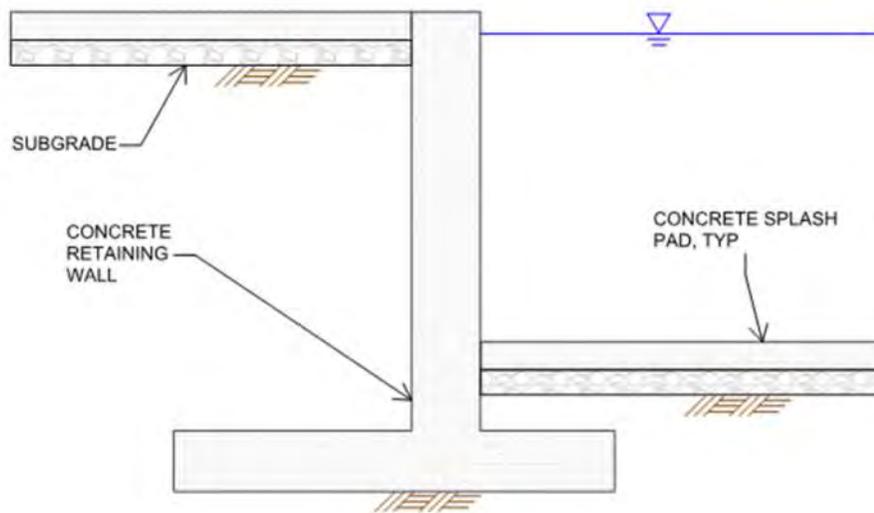


Figure 9 - Inverted-Tee Retaining Wall Section

### BENEFITS

The inverted-tee retaining wall has similar benefits to the floodwall such as shallow embedment, but the additional backfill on the retained side of the wall provides several advantages over the floodwall section. Note that a retaining wall would be considered a coastal structure by FEMA's classifications, not a floodwall. Coastal structures are not required to meet FEMA's freeboard requirements under 44 CFR 65.10.

Unlike the floodwall, a retaining wall has backfill on the landward side that adds redundancy and resistance to impact loads. Because of the added redundancy of the retaining wall system, the City of Corpus Christi instructed HDR to exclude impact loads because the soil on the retained side would

prevent a catastrophic failure of the system in the event of an impact, and the damage to the wall could be repaired after the storm.

The wall would be connected to the Promenade to the west and could become part of the City's Sports Entertainment and Art (SEA) district. In addition to the structural redundancy benefits, the backfill on the retained side of the wall would allow the triangular area landside of the floodwall to be developed for public use. The area could be developed into a small park alongside the ship channel, a meeting place for smaller public gatherings, etc.

#### CHALLENGES

As discussed in the geotechnical section of this document, it became apparent that there are several layers of weak soils, which present significant design challenges in the form of large settlement and very small allowable bearing pressures. Simply adding the soil on the landward side of the wall would cause, the pressure on the existing ground surface to exceed the allowable bearing pressure.

Because the allowable bearing pressures would be exceeded, the inverted-tee retaining wall section is unlikely to present a feasible design alternative. Piles would likely be required to ensure the wall would not experience problematic settlement nor exceed the allowable bearing pressures.

#### SHEET PILE RETAINING WALL

The final alternative considered was a steel sheet pile retaining wall, which rely upon embedment of the wall to laterally restrain the soils as compared to an inverted-tee retaining wall that uses bearing on soils. This design consists of individual, interlocking sheets which are driven to a set embedment and retain a soil fill on their backside. Sheet pile walls are commonly used in coastal environments for the design of bulkheads, canals, and harbors. Sheet pile retaining walls also carry the distinction of being labeled as coastal structures (by FEMA's classifications) and do not have associated FEMA freeboard requirements. There are several benefits to sheet pile wall systems with regards to structural, constructability, and public use considerations.

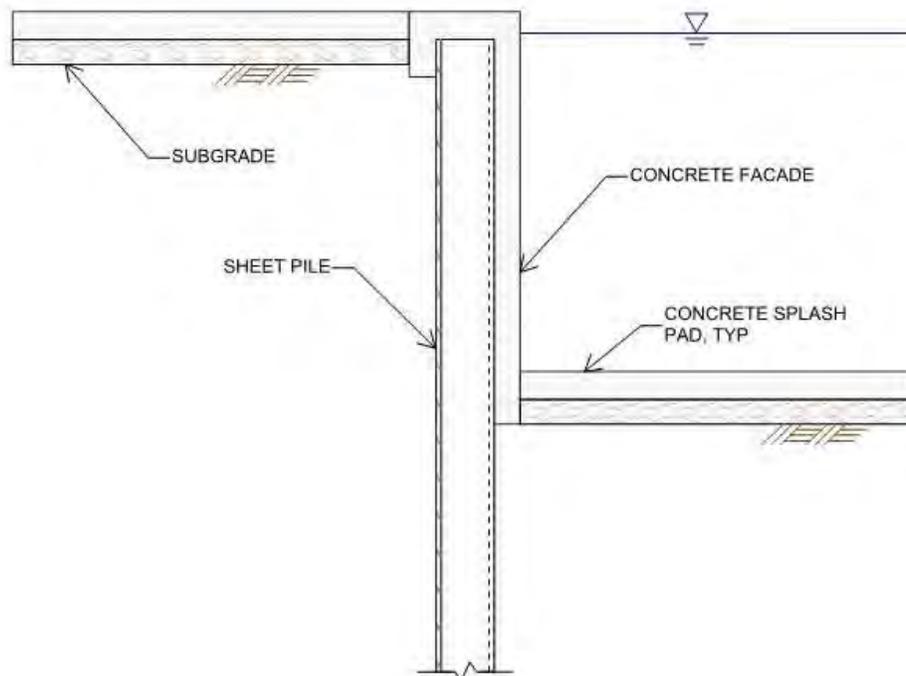


Figure 10 - Sheet Pile Retaining Wall Section

#### BENEFITS

A steel sheet pile wall is often encased by a concrete façade for durability protection. In addition to functionality, the concrete façade can be cast with a variety of patterns, colors and details which make it aesthetically pleasing. As discussed in the Inverted-tee retaining wall section, the triangular area encompassed by the concrete can be developed for beneficial use.

A sheet pile wall also provides benefits with regards to structural and constructability concerns. The sheet pile wall withholds a large soil mass which provides redundancy to resist impact loadings, including those of storm waves or accidental vessel impact. The sheet pile wall also does not require any excavation in order to place the wall.

#### CHALLENGES

The main challenge with a sheet pile wall is with regards to its required embedment, and any interference to which this may lead regarding existing utility lines. The issues with the utilities are detailed in the Impacted Utilities section of this document.

## SLOPE STABILITY

A slope stability analysis was performed for the sheet pile retaining wall concept. The analysis was performed using a software package named DeepEx. The HSDRRS design guidelines recommend using the spencer-price method of slope stability analysis, so they were used for this project.

Because the proposed sheet pile retaining wall is along the Corpus Christi Ship Channel, there are some sections of the sheet pile wall that are closer to the ship channel than others. Figure 11 and Figure 12

show the two sheet pile wall sections that were evaluated for the slope stability analysis. Figure 11 shows the sheet pile wall located closest to the ship channel, and is referred to as the wall section “On Slope.” Figure 12 shows the sheet pile wall that is located further from the ship channel and is referred to as the wall section “With Bench.”

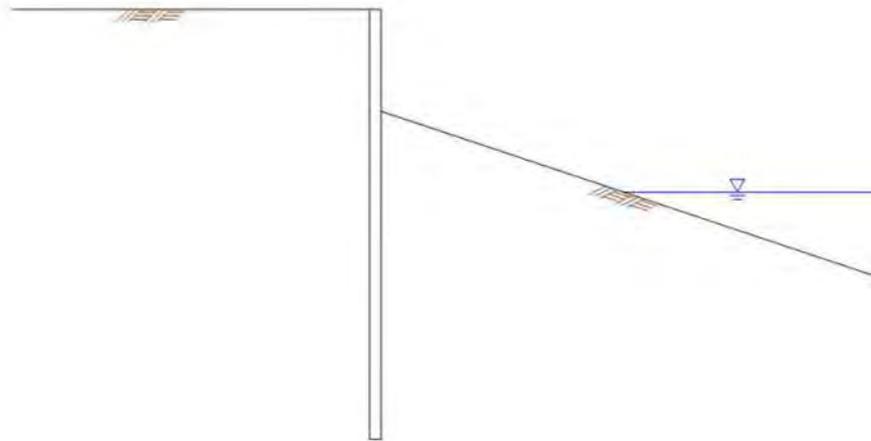


Figure 11 - Slope Stability Wall Section Diagrams, Wall near Slope



Figure 12 - Slope Stability Section Diagrams, Wall with Bench

Table 8 contains the results from the slope stability analysis. All of the cases provided slope stability factors of safety that met or exceeded the recommended factors of safety from the HSDRRS design guidelines.



**Table 8 - Slope Stability Analysis Summary**

Case	Soil Parameters	Wall Section	Soil Parameters	Actual FS	Required FS
Daily Loading	Drained	On Slope	Drained	1.50	1.5
		With Bench		1.88	
Construction Loading	Undrained	On Slope	Undrained	1.45	1.3
		With Bench		1.54	
Rapid Drawdown	Undrained	On Slope	Undrained	1.50	1.1
		With Bench	Undrained	1.64	
	Drained	On Slope	Drained	1.40	
		With Bench	Drained	1.47	

## WALL SIZE AND LENGTH CASES

In order to determine the required sheet pile section modulus and length of sheet piles, several load combinations were evaluated. The load cases listed in Table 9 were used to determine the required lengths of the sheet pile walls.

**Table 9 - Wall Design Cases**

Case	Soil Elevations <i>(ft, NAVD '88)</i>		Water Elevations <i>(ft, NAVD '88)</i>		Surcharge <i>(psf)</i>	Soil Parameters
	Retained	Excavated	Retained	Excavated		
Daily Loading	12.5	Near Slope	1.8	1.8	250	Undrained
	12.5	With Bench	1.8	1.8		
Construction Loading	12.5	Near Slope	12.5	1.8	75	Drained
	12.5	With Bench	12.5	1.8		
Rapid Drawdown	12.5	Near Slope	12.5	5.5	50	Undrained
	12.5	With Bench	12.5	5.5		
	12.5	Near Slope	12.5	5.5		Drained
	12.5	With Bench	12.5	5.5		
Wave Loading	12.5	With Bench	12.5	12.5	0	Drained

## FACTORS OF SAFETY

The following sections detail the factors of safety that were used for the design of the sheet pile wall.

### EMBEDMENT

The minimum factors of safety for determining penetration listed in Table 10 were taken from EM1110-2-2504, Design of Sheet Pile Walls and were used to determine lengths of the sheet pile walls.

**Table 10 - Minimum Safety Factors for Determining Penetration**

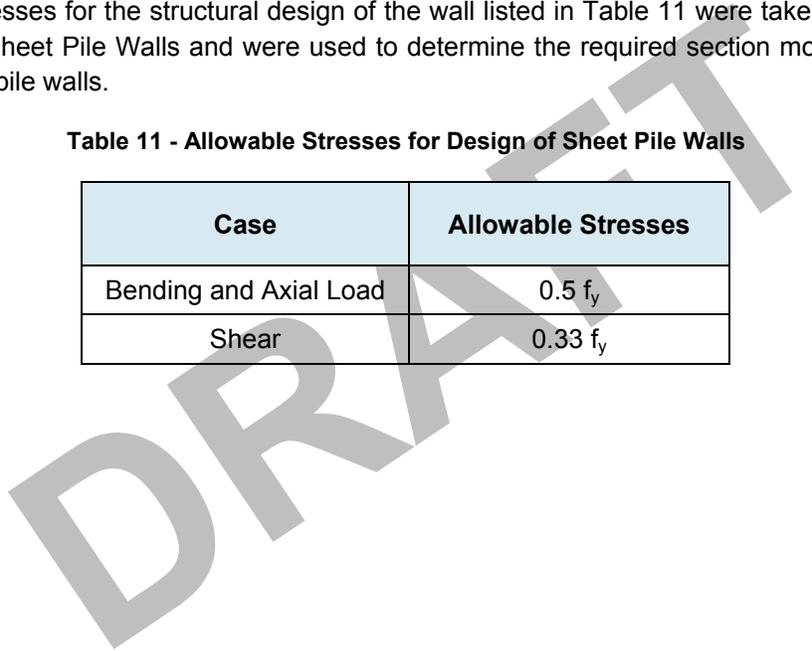
Case	Factor of Safety	Soil Conditions
Usual	2.0	Undrained
	1.5	Drained

**STRUCTURAL DESIGN**

The allowable stresses for the structural design of the wall listed in Table 11 were taken from EM1110-2-2504, Design of Sheet Pile Walls and were used to determine the required section modulus and area of steel of the sheet pile walls.

**Table 11 - Allowable Stresses for Design of Sheet Pile Walls**

Case	Allowable Stresses
Bending and Axial Load	0.5 $f_y$
Shear	0.33 $f_y$



## 6. USACE PERMITTING

U.S. Army Corps of Engineers (USACE) permitting considerations for construction of the proposed retaining wall include the following:

1. **Section 10/404 Regulatory Permit** – A preliminary review of recent and historical aerial photography and ground photos indicates that the retaining wall would be constructed entirely within uplands. No indications of jurisdictional wetlands were observed. However, in order to verify that jurisdictional wetlands are not present at the site, a formal site investigation is recommended. The investigation would include digging several test pits to verify that hydric soils, hydric vegetation, and hydrology are not present at the site. Results of the investigation should be submitted to USACE along with a letter requesting that construction of the retaining wall qualifies for a “no permit required” decision. The typical timeframe to receive a letter from USACE indicating that no permit will be required usually ranges between 1-3 months.
2. **Section 408 Review** – A Section 408 Review is USACE’s process that considers projects that may modify, alter, or occupy any existing USACE-constructed public works project. Public works projects include dams, basins, levees, channels, navigational channels, and any other local flood protection works constructed by the Corps. Since the proposed retaining wall would tie into an existing USACE constructed bulkhead, coordination with USACE regarding 408 Review will be necessary.

A 408 Review is initiated via a written request (Letter of Interest (LOI)) to the District. The District then forwards the LOI to the 408 Project Manager (PM) for processing. The 408 PM will identify the project as a Headquarters or District level request. The District PM will then determine if the project is subject to permission from USACE under Section 408. Should the proposed retaining wall need approval via Section 408, the District PM will decide whether the project is minor and/or routine and can qualify for Categorical Permissions request. If the District PM decides that the project is more complex and does not qualify for a Categorical Permissions request, a more extensive review process may be required. Based on initial review of the proposed retaining wall, it appears that the project will be considered minor and will likely qualify for a Categorical Permission request. The typical time frame for issuance of a Categorical Permission for a minor 408 project is 1-3 months. The typical time frame for issuance of a major 408 review is 12-18 months.

3. **Real Estate Outgrant** – Coordination with the USACE Real Estate Department will be necessary since the proposed retaining wall is likely located within the USACE Navigation Easement or potentially within a Federally-controlled land tract. Should a Section 10/404 regulatory action be initiated, the Real Estate Department will have an opportunity to review the project for Federal real estate interests at that time. In order to determine whether the proposed retaining wall falls within an area of Federal interest, the USACE Real Estate Department will request that CAD or GIS shape files of the retaining wall be submitted. The project footprint will then be uploaded into a USACE mapping database which will show its’ location relative to areas of Federal interest. Should the project be located within the Federal interest, a USACE Real Estate Outgrant application and corresponding permit drawings should be submitted. The result of the Real Estate process is issuance of a Real Estate license/outgrant for the proposed project. The typical time frame to receive a Real Estate Outgrant (once application is submitted) is 12 months.



## 7. OPINION OF PROBABLE PROJECT COST

A preliminary Opinion of Probable Project Cost (OPPC) was developed for the proposed sheet pile retaining wall described in Section 5. The OPPC is based on HDR's understanding of current labor and material costs and reasonable estimates of construction activity duration. Improvements included selective demolition, pre-loading soils at the site, constructing a sheet pile retaining wall, and paving. As shown in the summary in Table 12, the OPPC for the proposed concept is approximately \$1,421,700, which includes construction and professional services. Permitting and final design would likely require approximately 12 to 16 months, and construction would likely require approximately 18 months

**Table 12 - OPPC for Sheet Pile Retaining Wall**

Items	Description	Total
A1	Mobilization / Demobilization	\$ 50,000
A2	Demolition	\$ 81,000
A3	Earthworks / Site Grading	\$ 53,000
A4	Wall and Misc. Improvements	\$ 735,000
	<b>Sub Total</b>	<b>\$ 919,000</b>
B1	Other Project Related Costs	\$ 227,000
C1	Contingency (30% of Subtotal)	\$ 275,700
	<b>Project Total</b>	<b>\$ 1,421,700</b>

### BARGE IMPACT COSTS

A conceptual-level analysis was performed to determine the order of magnitude potential increase for including and not including impacts from loose barges during a storm. Table 13 shows the relative cost differences for the wall and the concrete caps with and without designing for barge impacts.

**Table 13 - Barge Impact Comparison**

Items	Description	Barge Impacts	Total
COMP1	Steel Sheet Piling with Concrete Cap	No Barge Impact	\$ 322,000
COMP2	Steel Pipe Piling with Concrete Cap	Incl. Barge Impact	\$ 1,562,000
	<b>Magnitude of Increase</b>		<b>485%</b>

Based on this analysis, the cost of the proposed wall would increase by approximately 485% if barge impacts are included in the design of the wall. It should be noted that conceptual level calculations were performed to determine the size of wall required to incorporate barge loads. The conceptual level calculations relied upon the retained soil to resist the impact load from the barge. In the HSDRRS guidance for impact loads, the barge extreme case only needs to ensure that the wall does not collapse. Therefore, a wall section was determined making sure the wall section does not collapse, but the wall would likely sustain some repairable damage under a barge load that could be addressed after a storm.

During a meeting with the City on July 7, 2016, designing the retaining wall to withstand barge impact loads was decided to be unnecessary after considering the high additional cost, and the available redundancy against flooding that would be offered by the large mass of soil landward of the wall.

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## 8. REPORT LIMITATIONS

- This report was developed by HDR for the client's explicit use. Use of this work product by others is at their own risk. The content included in this report is correct to the best of our knowledge and has been developed in accordance with the standard of care that is typical a practitioner in this industry. The standard of care was followed for collection and analysis of data, and for calculations or modeling performed in support of this report.
- This report is not meant to contain an exhaustive or complete evaluation of all potential or possible design alternatives. Any decisions that are made on the basis of this report are the responsibility of the owner. Decisions by the owner should take into account the limitations and residual risks identified or documented in this report. HDR does not warrant or guarantee our work or recommendations.
- Some of the information provided in this report was developed or provided by others. Except as specifically identified within this report, HDR has not performed independent validation or verification of exploration data, modeling data, or other analysis on data provided by others.
- The conclusions and recommendations in this report are based on geotechnical conditions of the project site at the time of this study. Any modifications to the site, man-made or natural, could alter the analysis, findings, and recommendations contained herein and could result in the analysis, findings, and recommendations to no longer be valid. Additional work may be required as changes occur or new information becomes available.
- The conclusions and recommendations in this report are based on the conditions of the project site at the time of this study. Any modifications to the site, man-made or natural, could alter the analysis, findings, and recommendations contained herein and could result in the analysis, findings, and recommendations to no longer be valid. Site conditions, climate changes, vegetation changes, maintenance practice changes, or other factors may change over time. Additional analysis or updates may be required in the future as a result of these changes. Parties other than the client for whom this work was developed under contract, must notify HDR if they would like to use this report for any purpose. HDR may request that additional work be performed and that an updated report be issued.
- Water surface elevations were evaluated for the purposes of potential future accreditation of the proposed retaining wall as a coastal structure under the Federal Emergency Management Agency (FEMA) National Flood Insurance Program (NFIP). FEMA has established minimum design criteria for accrediting coastal structures for the purposes of administration of the NFIP for the 100-year event. Accreditation does not mean a coastal structure provides adequate damage/risk reduction for a particular community. Additional studies and modifications of a coastal structure system could be warranted to provide a higher and more appropriate level of flood risk mitigation.
- Certification does not assure that the coastal structure will protect against future flood events. Even with a certified flood protection system in place, a possibility of flooding caused by overtopping or other failure modes exists. Floodplain management measures to reduce the consequences of this possibility, such as elevating structures, maintaining a current flood warning system and evacuation plan, and wisely managing floodplain development are strongly advised.

## 9. ITEMS PENDING FOR FINAL DESIGN

Following the City's review of this memorandum, HDR will submit preliminary (30% level) construction plans, with final design to be performed under a future contract or task order.

- Additional coordination with the City's planning department is needed to identify possible beneficial uses for the area landward of the retaining wall. The plans for this space will determine the need for public-use features such as ramps, benches, walkways, etc.
- The location of the High Tide Line (HTL) needs to be determined to verify that the splash pad would not be located within jurisdictional waters of the U.S., and an associated Section 10/404 permit is not needed from USACE.
- Details need to be developed for the scour protection system near the west end of the proposed retaining wall.
- Additional analysis is needed for characterization of the consolidation and settlement of the fill material that would be placed landward of the proposed retaining wall. In particular, the time required for the pre-loading needs to be evaluate in more detail

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