HDR

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11/30/16

# Memo

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| Date: | Wednesday, November 30, 2016 |
| Project: | Salt Flats Levee System, Phase 2 (Project No. E12070) |
| To: | Daniel Deng, P.E.; Jeff Edmonds, P.E. |
| From: | Curtis Beitel, P.E., CFM; Brandon Hilbrich, P.E., CFM; Dan Heilman, P.E. |
| Subject: | Task Order No. 3 – Final 2D Hydraulic Model of the Downtown Seclusion Area |

Under a previous effort referred to as “Phase 2A” of the overall Salt Flats Levee assessment, HDR performed a preliminary review of existing available information and developed recommendations for improvements to the Salt Flats Levee. Under Task Order No. 2 potential vulnerabilities were assessed in the other eight elements of the City of Corpus Christi’s (City’s) overall Downtown Flood Protection System, resulting in five overall strategies, one of which was to develop a more detailed 2D hydraulic model of the downtown area, which FEMA shows as a “Seclusion Zone” on the Preliminary FIRM maps. As stated in FEMA’s 2015 Levee Analysis and Mapping Procedure (LAMP) Plan for Nueces County, the City agreed to apply the seclusion method for areas that may be vulnerable to flooding depending on the level of protection provided by the Turning Basin Tributary Levee (i.e., Salt Flats Levee). The Seclusion designation would allow the rest of the County’s updated FIRM data to be finalized and adopted without the levee mapping revisions or levee certification, which would be incorporated in the future. The Seclusion Zone can be seen in **Figure 1**. Regardless of how the seclusion area will eventually be mapped by FEMA, the City desires clarity on the level of protection provided by the existing levee and related features of the overall downtown flood protection system.

As documented herein, Task Order No. 3 focused on development of a detailed hydraulic model for the downtown area to evaluate the City’s flood protection system. Specifically, the modeling allows more detailed assessment of the effectiveness of the existing pumping system, and more accurate estimates of flood levels during different storm events. The model considers contributions from rainfall, storm surge, and wave overtopping sources.

The scope of services under Task Order No. 3 is:

1. Initiation and Controls: General project management duties such as status reporting, scheduling of manpower and project deliverables, staff assignments, internal coordination meetings, deliverables and quality control (QC).
2. Survey Finished Floor Elevations: Perform a field survey to obtain the lowest adjacent grades of buildings and residential homes at approximately 20 selected locations within the Seclusion Zone. Prior to fieldwork, prepare an exhibit of proposed locations and meet with the City to discuss which buildings and homes will be surveyed. Summarize survey results on a map of the Seclusion Zone.
3. Refine 2D Model of Existing Conditions: Develop a 1D/2D coupled XPSWMM model for existing conditions:
    a. Hydrology Update: Runoff from the interior drainage areas is pumped out by two storm water pump stations. Start with the drainage areas developed as part of the Pump Station designs and verify the boundaries of the drainage areas contributing to the Seclusion Zone. Update the hydrologic parameters using GIS and provided landuse

datasets to calculate the flow hydrographs to the contribution points in the system. Develop updated hydrology for the 10%, 4%, 2%, and 1% annual chance events (ACEs) based on the precipitation from the U.S. Geological Survey's SIR 98-4044. Verify the area of interior flooding using guidance found in the USACE Engineering Manual 1110-2-1413, Hydrologic Analysis of Interior Areas.

   b. Seawall Overflow Contribution: Using the results of the coastal wave modeling previously performed by HDR under City Project E12070, estimate the duration and rate of wave overtopping over the Seawall along Shoreline Boulevard during the 1% ACE[1] based on the storm surge model prepared for FEMA's 2011 TSDN for Nueces County. Calculate wave overtopping rates with and without allowances for future (50 year and 100 year) estimates of sea level rise based on the USACE sea level rise calculator. Apply the overtopping flow hydrographs along the edges of the XPSWMM model terrain.
   c. Hydraulic Model: Develop a 1D/2D coupled XPSWMM model using the USGS 2011 LiDAR surface, available storm drain record drawings and pump station details. Model hydraulically significant stormwater conveyance systems, such as stormwater pipes that facilitate flow to the pump stations. Model each pump separately, with its pump curve and on and off elevations.
      i. Facilitate a workshop with Urban Engineering and City staff to review the 1D/2D XPSWMM existing conditions model. Review the resulting inundation areas for existing conditions and develop a set of desired Seclusion Area constraints.
   d. Interior Hydrology Technical Memorandum: Prepare a technical memorandum describing the development and refinement of the 1D/2D model, including exhibits of the proposed Seclusion Area inundation extents. Incorporate City review comments and provide an updated technical memorandum to submit to FEMA as part of the LAMP Pilot Study process, if desired by the City.

4. Pump System Alternatives Analysis: Starting with the set of desired Seclusion Area constraints from the workshop with City staff and evaluate the following scenarios:
   a. Blucher Arroyo Pressure Box Contribution: Using the updated hydrology, estimate the duration and rate of additional flow contribution from a hypothetical failure of the Blucher Arroyo Pressure Box during the 1% ACE.
   b. Proposed Hughes Street Pump Station: Urban Engineering has also completed the design of a third pump station at Hughes Street and associated improvements, referred to as the Downtown Drainage improvements, Phase III, City project 2226 A&B. Add the additional pump station into the 1D/2D XPSWMM model to quantify the additional benefit for the 1% ACE.
   c. Evaluate up to three potential improvement scenarios with the 1D/2D XPSWMM model to help determine necessary pump station modifications to achieve the desired Seclusion Area extents for the 1% ACE.
   d. Review Meeting: Facilitate a meeting with the City to review the results of the pump systems alternatives analysis.
   e. Final Memorandum: Include results in the Interior Hydrology Technical Memorandum described under Task 3.d, to finalize the memorandum.

[1] 1% ACE still water elevation is below the Seawall crest. Seawall overflow contribution is limited to wave overtopping. No overtopping would occur from a surge-only (without waves) condition.

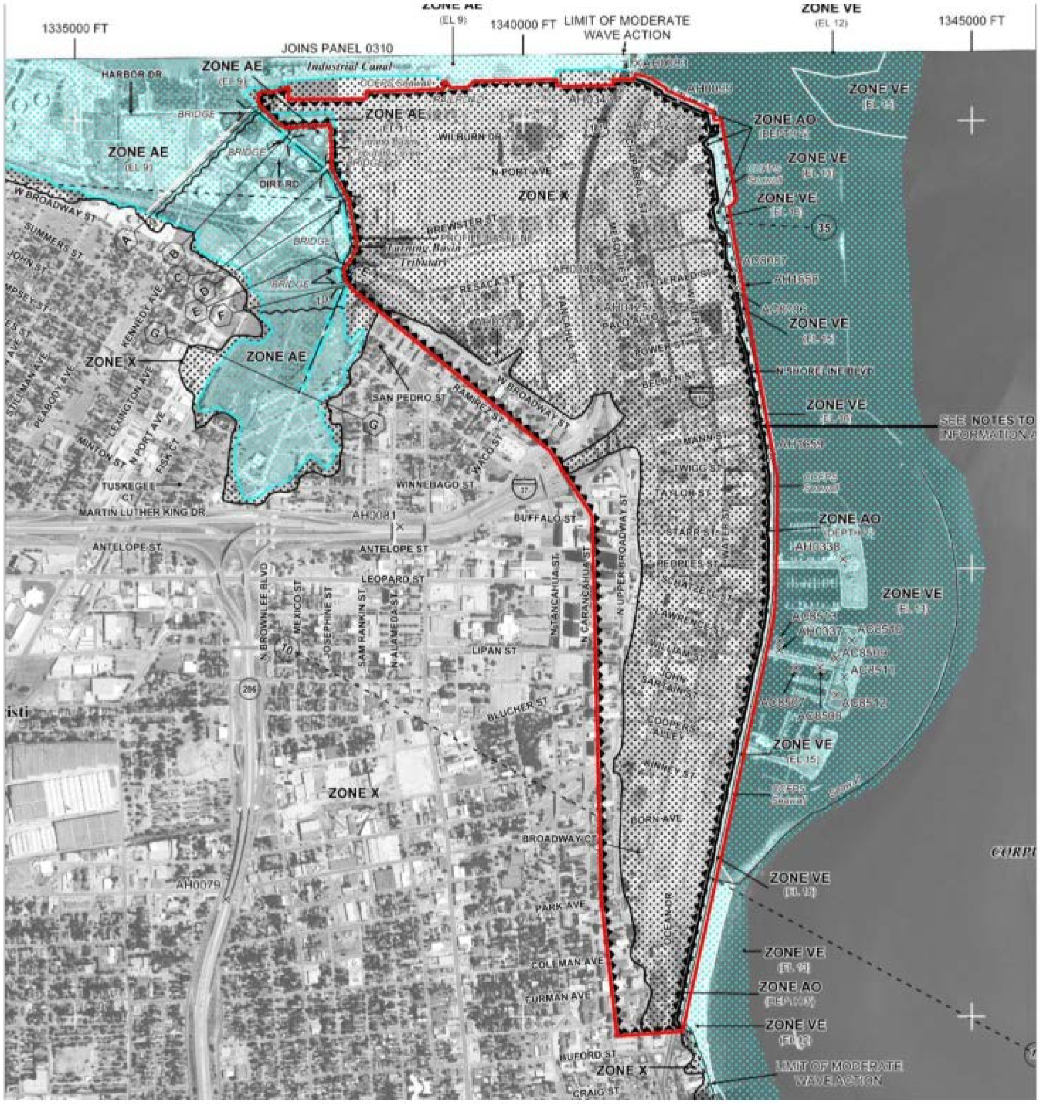


**Figure 1 – Seclusion Zone as defined by FEMA**

# 1 Kickoff Meeting

A kickoff meeting for Task Order No. 3 was held at the City's Engineering Department on May 23, 2016 to review the City's goals and discuss project scope and schedule. Details of the modeling approach were discussed, and data needs and availability were reviewed. The City emphasized that they are not necessarily seeking system certification for compliance with the National Flood Insurance Program. Although several of the system components (e.g., Salt Flats Levee) are expected to be mapped by FEMA as being freeboard deficient, the City's goal is for the components to be structurally sound. Preventing the downtown area from being mapped as a Flood Zone is not a goal.

# 2 Study Area

For this task order, a study area of approximately 4 square miles within the Inner Harbor Basin (defined in the City of Corpus Christi Storm Water Master Plan) defined the watershed contributing to the major Downtown storm drain and Salt Flats drainage systems as shown in **Figure 2**. The watershed is highly urbanized with some medium to high density residential areas west of the downtown area. Approximately 60% of the study area is conveyed through major storm drain systems that outfall to the Salt Flats tributaries and, ultimately, the Corpus Christi Turning Basin portion of Corpus Christi Inner Harbor. The remaining area is conveyed through several storm drain systems east through the Downtown area, collecting in a low lying area that is drained by two stormwater pump stations out to Corpus Christi Bay.


**Figure 2 – Project Study Area**

# 3 Existing System Overview

As shown in **Figure 3**, the Downtown Flood Protection System consists of several elements, including (from east to west) the earthen Salt Flats Levee (No. 1), the Port Authority Wharf (No. 2), Promenade (No. 3), Museum Floodwall (No. 4), U.S. Army Corps of Engineers (USACE) Bulkhead (No. 5), South Art Museum and Barge Dock (No. 6 and 7) and City Seawall (No. 8), as well as two storm water pump stations located at Kinney Street (No. 9) and Power Street (No. 10) and two major storm drain pressure box systems along John Sartain Street (No. 11) and Kinney Street (No. 12). **Figure 4** shows additional detail for the northern portion of the Downtown Flood Protection System from the Salt Flats Levee to the Seawall.


**Figure 3 – Corpus Christi Downtown Flood Protection System**


**Figure 4 – Port Authority Wharf to City Seawall**

# 4 Collection and Review of Available Information

HDR gathered and compiled readily-available historical information, including geospatial data, engineering drawings, as-built plans, and previous studies, to help characterize the structural make-up and limits of the various system elements within the study area. All data were provided as digital files in geographic information system (GIS) or PDF format.

A complete list of all data collected and their sources is included in **Attachment A**. All digital files, including plan pdf files and geospatial data, are included in the submittal to the city.

## 4.1 Topography

Available LiDAR topographic data previously collected and processed in 2011 were acquired from the City. The 1-foot contour data and digital elevation model (DEM) data were provided in the North American Datum of 1983 (NAD 83) State Plane Texas South feet (horizontal) and North American Vertical Datum of 1988 (NAVD 88) datum.

Similar to FEMA's 2015 Levee Analysis and Mapping Procedure (LAMP) Plan for Nueces County, this LiDAR dataset was used for the hydrologic and hydraulic analyses performed in this study[2]. For additional information on LiDAR collection methods, accuracy, and metadata, refer to Section 7 of the 2015 LAMP report and Appendix E.

## 4.2 Soils

The latest Soil Survey Geographic Database (SSURGO) data (2008) for Nueces County was downloaded from the NRCS Web Soil Survey online repository. The entire study area is completely contained within

[2] LiDAR dataset used to define 30-foot x 30-foot grid for 2D hydraulic model.

the Urbanland (UA) soil type. Additional discussions on soil types are included in the Section 5.2 Hydrology.

## 4.3 Land Use

The City maintains a land use dataset which is available for viewing on the City of Corpus Christi GIS Map Viewer. The current landuse dataset was extracted and provided by the City for use in this study. The detailed landuse dataset includes eighteen landuse types throughout the city. Within the study area, fifteen of those landuse types existed with the most prevalent uses being public/semi public and street areas due to the downtown urban setting. **Table 1** summarizes all land use types included in this study and the approximate total acreage of each. Additional discussions on landuse types are included in Section 5.2 (Hydrology).

**Table 1 – Landuse Types within Study Area**

| Landuse Type | Symbol | Area (ac) | Landuse Type | Symbol | Area (ac) |
|---|---|---|---|---|---|
| Commercial | COM | 192.4 | Mobile Homes | MH | 9.4 |
| Drainage Corridor | DC | 8.1 | Park | PARK | 138.0 |
| Estate Res. | ER | 8.0 | Professional Office | PO | 68.2 |
| High Density Res. | HDR | 10.8 | Public/Semi-Public | PSP | 357.5 |
| High Industrial | HI | 34.8 | Streets | Road | 671.3 |
| Low Density Res. | LDR | 314.1 | Railroad ROW | ROWRR | 11.0 |
| Light Industrial | LI | 243.3 | Vacant | VAC | 331.7 |
| Medium Density Res. | MDR | 135.1 | | | |

## 4.4 Building Planimetrics

CyberCity 3D, a geospatial modeling company specializing in measurements of 3D buildings from stereo imagery, was hired by the Downtown Management District to develop 3D building planimetrics for the downtown area. As part of this study, building planimetric data were provided by the City for use in the 2D computation of the hydrologic and hydraulic models. Data were provided as a multipatch (3D object) geospatial dataset and converted to 1-dimensional polygon shapefiles.

In addition to the downtown area, building planimetrics within the Salt Flats and Blucher Arroyo watersheds were created using available aerial photography to provide coverage over the entire area included in the 2D computation of the hydrologic and hydraulic models.

The building planimetric data can be seen on **Exhibit B1** in **Attachment B**. Applicability of data within the hydrologic and hydraulic models is further discussed in Section 5.4 (Hydraulic Model Development).

## 4.5 Record Drawings

Through coordination with the City and Urban Engineering, as-built drawings were acquired for the majority of the drainage infrastructure within the study area including major storm drain systems, Salt Flats Channel, Power Street and Kinney Street pump stations, and Seawall related project improvements. The vertical datums associated with each as-built set varied between NGVD 29 and NAVD 88. As previously stated, all modeling and mapping efforts were completed in the NAVD 88 vertical datum. According to the National Geodetic Survey (NGS) Vertcon Progr B3.2am, Version 2.1 (VERTCON), the datum conversion from NGVD 29 to NAVD 88 is -0.62 foot which agrees with the -0.64 foot listed in Table 8 of the Nueces County Preliminary FIS Report.

All as-built plans received are summarized in **Table A1** in **Attachment A**.

## 4.5.1 Storm Drains

GIS data for the existing storm drain systems were acquired through the City. Separate GIS shapefiles were provided for stormwater ditches, storm drain mains (and laterals), grate inlets, curb inlets, and manholes. These data were utilized for the determination of the “major” drainage systems throughout the Downtown and Salt Flats areas. All pipes 30” in diameter or larger were considered to be major conveyance systems in the areas along the floodwalls and seawalls (i.e., Port Area, Power Street, Kinney Street, Coliseum, and Furman watersheds). Additionally, several minor systems 24” in diameter that outfall directly into Corpus Christi Bay along the Seawall were included to appropriately drain flooding along the Downtown low lying areas. Only the main trunk lines and collection points were considered “major” within the Blucher Arroyo and Salt Flats watersheds. **Table 2** summarizes the seven major drainage systems to be included in the model for this study. The major drainage systems were named according to the watersheds they fell within.

**Table 2 – Major Drainage Systems**

| Storm Drain System | System Description |
|---|---|
| Blucher Arroyo | Marguerite Street, S Staples Street, and N Carrizo Street storm drains collect in an open channel at Blucher Park and outfall to the Bay through a 9’ horseshoe pressure pipe between John Sartain Street and Coopers Alley. |
| Coliseum | A 36” storm drain along Broadway Court outfalls to the Bay just north of Park Avenue. |
| Furman | Furman Avenue and Buford Street storm drains combine to outfall into the Bay through a 54” pipe at Furman Avenue. |
| Kinney Street | Water Street and Kinney Street storm drains convey collected stormwater to the Kinney Street pump station. |
| Port Area | Storm drains along Port Avenue, N Mesquite Street and Hirsch Street outfall through the floodwall. |
| Power Street | Hughes Street, IH 37 and Power Street storm drains connect to the horseshoe pipe along Water Street which conveys collected stormwater to the Power Street pump station. |
| Salt Flats | Storm drains along 19$^{th}$ Street and Port Avenue collect in an open channel near Lipan Street southeast of the IH 37/US 286 intersection; this system along with the John Street storm drain contribute to the Salt Flats Channel which flows northward and empties into the Corpus Christi Inner Harbor. |

**Exhibit B1** in **Attachment B** displays the major storm drain systems incorporated into the hydrologic and hydraulic model for this study. Data for the major drainage systems, such as conduit material, shapes, sizes, and invert elevations, were obtained through record drawings when available. When record drawings were not available, the City stormwater GIS files were referenced for the specific conduit data.

## 4.5.2 Pump Stations

Two existing pump stations located within the Kinney Street and Power Street watersheds pump stormwater runoff from the Downtown area and were incorporated into the hydraulic model for this study.

Pump characteristics and operations for the Kinney Street pump station referenced the Downtown Drainage Improvements Phase 1 Project B – Kinney Street Pump Station design memorandum, dated June 2004, with supplemental information received from Urban Engineering.

The Kinney Street pump station is located at the intersection of Water Street and Kinney Street. The pump station conveys stormwater from the 8'x5' box and 36" pipe storm drain systems located along Water Street north and south of the pump station, respectively. The pump station also receives flow from a 24" pipe storm drain along Kinney Street entering from the east of the pump station. For the hydraulic model, the operational pump station configuration (two sumps and four pumps) and wet well volume were taken from the Downtown Drainage Improvement plans for the Kinney Street Pump Station dated 2008. The fifth submersible pump shown in the design plans acts as a backup to the other four pumps during normal operation.

A report showing the Flygt factory flow test results for the five P7121 submersible pumps was provided by Urban Engineering. Plots of the test results for total dynamic head vs. flow rate vs. input power were included in the report for each of the five pumps. Rather than applying a unique pump curve to each of the pumps in the hydraulic model for this study, a single "combined" curve was developed by plotting the factory test results and fitting a second order polynomial line to the data. Therefore, the four pumps in the hydraulic model referenced this combined pump curve as shown in **Table 3** below. As mentioned above, only four of the five pumps are used during operation and were incorporated into the hydraulic model.

A Flygt factory flow test (**Table 4**) for the NP3153 sump pump (provided by Urban Engineering) was referenced for the pump curve data associated with the two small supplemental sump pumps operating to keep the wet well dry.

**Table 3 – Kinney Street Pump Curve**

| TDH (ft) | Single Pump Flow | |
|---|---|---|
| | (gpm) | (cfs) |
| 30 | 39,412 | 87.82 |
| 28 | 42,155 | 93.93 |
| 26 | 44,731 | 99.67 |
| 24 | 47,141 | 105.04 |
| 22 | 49,384 | 110.04 |
| 20 | 51,461 | 114.66 |
| 18 | 53,371 | 118.92 |

*TDH: Total Dynamic Head*

**Table 4 – Kinney Street Sump Curve**

| TDH (ft) | Single Pump Flow | |
|---|---|---|
| | (gpm) | (cfs) |
| 86 | 0 | 0.00 |
| 80 | 150 | 0.33 |
| 75 | 250 | 0.56 |
| 70 | 375 | 0.84 |
| 65 | 525 | 1.17 |
| 60 | 675 | 1.50 |
| 55 | 875 | 1.95 |
| 50 | 1,050 | 2.34 |
| 45 | 1,190 | 2.65 |
| 40 | 1,350 | 3.01 |
| 30 | 1,625 | 3.62 |
| 20 | 1,900 | 4.23 |
| 12 | 2,125 | 4.73 |

*TDH: Total Dynamic Head*

Pump operations for the Kinney Street pump station were obtained through coordination with Urban Engineering and the Kinney Street pump station operations City personnel. **Table 5** summarizes the pump operations for the Kinney Street pump station. Due to limitations within XPSWMM pump controls, starting elevations must be higher than stopping elevations. Therefore, the pump operation trigger elevations were modified so pumps would operate as intended. The modified pump operations, as shown in **Table 5**, were included to hydraulically simulate pump effectiveness within the hydraulic model.

Based on the normal operation of the sump pumps, the wet well was assumed to be dry initially (depth of 0 feet), which equated to the wet well invert of -18.5' NAVD 88. An initial depth of zero accounted for the activity of the two sump pumps.

**Table 5 – Kinney Street Pump Operations**

| Pump | Pump Status | Elevation NAVD 88 (ft) | Mod. Elevation NAVD 88 (ft) | Wet Well Depth (ft) |
|---|---|---|---|---|
| Lead Sump | On | -17.50 | -17.50 | 1.00 |
| | Off | -16.50 | -18.50 | 2.00 |
| Lag Sump | On | -16.50 | -16.50 | 2.00 |
| | Off | -18.50 | -16.51 | 0.00 |
| Pump 1 | On | -8.50 | -8.50 | 10.00 |
| | Off | -6.50 | -11.00 | 12.00 |
| Pump 2 | On | -7.50 | -7.50 | 11.00 |
| | Off | -7.50 | -8.50 | 11.00 |
| Pump 3 | On | -6.50 | -6.50 | 12.00 |
| | Off | -8.50 | -7.50 | 10.00 |
| Pump 4 | On | -5.50 | -5.50 | 13.00 |
| | Off | -11.00 | -6.50 | 7.50 |

**Figure 5** graphically shows the original pump operations versus the modified operations.

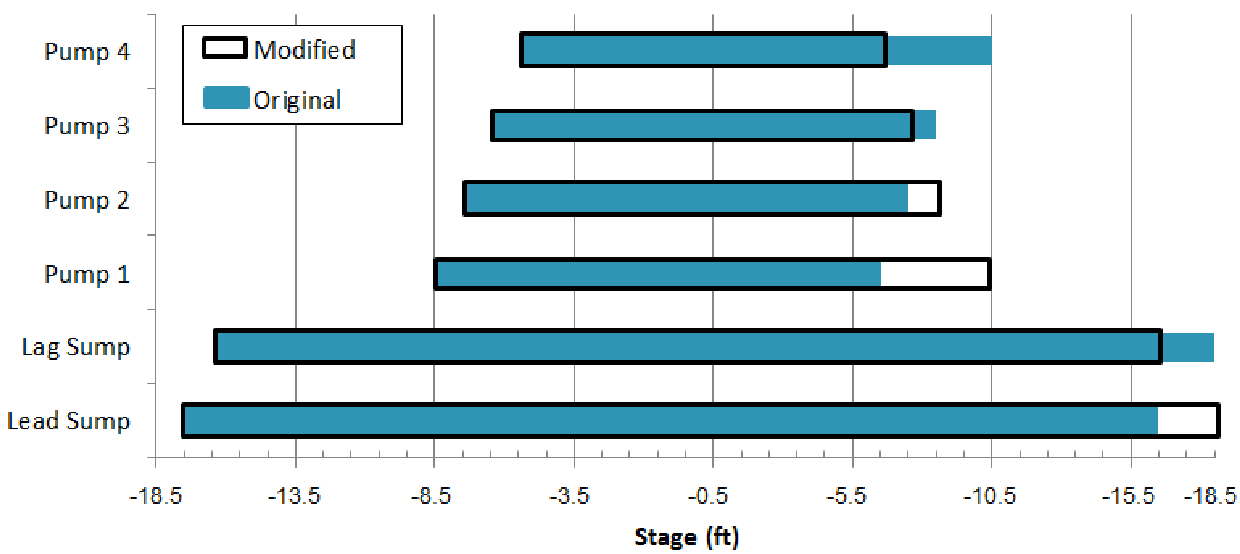


**Figure 5 – Kinney Street Pump Operations Schematic**

The Power Street pump station is located at the intersection of Water Street and Power Street. The pump station conveys stormwater from the 7' horseshoe and 9' horseshoe storm drain systems located along Water Street north and south of the pump station, respectively. For the hydraulic model, the pump station configuration (three pumps) and wet well volume were taken from the Bay Front Protection Detail Plans dated 1947. Through coordination with Urban Engineering and the City's Power Street operations personnel, it was determined the pump replacement in the 2005 contract documents was not implemented. Additionally, two of the original pumps were refurbished and the pump from the old Kinney Street pump station was moved to the Power Street pump station in 2006. Therefore, the three pumps in the hydraulic model referenced the pump curve included in the "Engineering Investigation/Analysis for Downtown Flooding of April 3, 1997" dated June 1997 and as shown in **Table 6** below. All three pumps are used during operation and were incorporated into the hydraulic model.

**Table 6 – Power Street Pump Curve**

| Total Dynamic Head (ft) | Single Pump Flow (gpm) | Single Pump Flow (cfs) |
|---|---|---|
| 12 | 66,750 | 148.73 |
| 11 | 70,750 | 157.64 |
| 10 | 73,750 | 164.33 |
| 9 | 76,250 | 169.90 |
| 8 | 78,750 | 175.47 |
| 7 | 80,750 | 179.92 |

Provided pump operations for the Power Street pump station were modified and included to hydraulically simulate pump effectiveness within the hydraulic model. **Table 7** summarizes the pump operations for the Power Street pump station. Based on the operations, an initial depth of 3.92' was assumed within the system. With a wet well invert of -10.83' NAVD 88, this equated to an initial elevation of -6.91' NAVD 88.

**Table 7 – Power Street Pump Operations**

| Pump | Pump Status | Elevation NAVD 88 (ft) | Wet Well Depth (ft) |
|---|---|---|---|
| Lead Pump | On | -4.26 | 6.57 |
| | Off | -6.91 | 3.92 |
| Lag 1 Pump | On | -3.99 | 6.84 |
| | Off | -5.92 | 4.91 |
| Lag 2 Pump | On | -3.76 | 7.07 |
| | Off | -5.47 | 5.36 |

In 2004, the 8'x5' box system along Water Street mentioned above was constructed from Kinney Street to Williams Street which connected to the existing storm drain system draining to the Power Street pump station. Therefore, the two pump stations became hydraulically connected by this system which allows conveyed stormwater to discharge through either pump station depending on available capacity. The new Water Street system referenced the "Downtown Drainage Improvements Phase 1 – Project A. New Interceptors and Inlets", dated October 2004.

See **Exhibit B1** for locations of pump stations with respect to the interior drainage system.

### 4.5.3 Projects in Design

The Texas Department of Transportation is in the early stages of design for the Harbor Bridge relocation project. The proposed project will realign Hwy 181 and Harbor Bridge within the Downtown area. Based on the current bridge design, the project would require realignment of the Salt Flats channel, possibly impacting drainage patterns within the surrounding area. Because the improvements will be located within the FEMA regulated Special Flood Hazard Area (SFHA), the bridge designers are planning on requesting a Conditional Letter of Map Revision (CLOMR) from FEMA. The CLOMR effort will involve hydraulic modeling to assess potential floodplain impacts from the bridge relocation project. Consideration of potential impacts from the Harbor Bridge relocation project was not included in the scope for the current hydraulic modeling investigation.

### 4.5.4 Projects in Construction

During the data acquisition process, it was determined an active project was currently under construction within the study area. The South Stapes Street improvements project was released for construction in May 2015. The project includes improvements to South Stapes Street from Morgan Avenue to IH 37 along with extensive storm drain system improvements. The new upsized storm drain is a series of large pipe and box storm drains that contribute to different collection points in the Blucher Arroyo system while remaining completely connected through several equalization pipes. The system flows southward from IH 37 and connects to the 24” pipe that flows east on Kinney Street and conveys flow to the north end of Blucher Park. Continuing to flow south from Kinney Street, a single 5’x4’ box culvert connects to 2-6’x4’ boxes in the Blucher Arroyo system just north of Marguerite Street The Staples Street storm drain flows northward from Morgan Avenue and connects to the 60” pipe along Marguerite Street which conveys flow toward the south end of Blucher Park.

Construction is approximately 40% complete with an initial estimated completion date of September 2017. Due to the location of the project within the Blucher Arroyo watershed and the potential impacts to the downstream Blucher Arroyo system, the proposed improvements were included in the existing conditions model or base scenario.

## 4.6 Previous Studies

The City is currently working with Goldston Engineering (CH2M Hill) to develop a city-wide storm water master plan (SWMP) to establish standard principles for the analysis, design, and construction of drainage infrastructure within the city. Preliminary system maps and supporting calculation documentation are available on the City of Corpus Christi website. Collected as-built data was supplemented with information from the master plan to complete the hydraulic model for this study.

# 5 Hydrologic and Hydraulic Analysis Approach

The software program XPSWMM 2014 Service Pack 1 was used to develop a dynamic 1D/2D rainfall-runoff model (accounts for timing and volume) for the 4 square mile study area. This software was chosen for its ability to simulate combined effects of complex overland drainage patterns, underground storm drainage systems, channels, and pumps. It was used to establish interior flood inundation limits for the Downtown area based on the defined design scenarios discussed in this section.

The development of the 1D/2D hydrologic and hydraulic model includes four overall components to analyze the existing drainage infrastructure during storm conditions. The following components were used to create the model:

1. Establish modeling and design criteria;
2. Develop hydrology to calculate surface flows at each inflow node of the model;
3. Estimate additional surface flow volume contributed by the wave runup and overtopping of the Seawall; and
4. Catalog and incorporate major storm drains, channels, and pump stations.

## 5.1 Modeling and Design Criteria

Hydrologic and hydraulic analyses were performed for the 10%, 4%, 2%, and 1% ACE over a 24 hour duration in accordance with FEMA criteria and based on the precipitation statistics in the U.S. Geological Survey's (USGS) Scientific Investigation Report 98-4044, "Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas", also referenced in the City of Corpus Christi Drainage Design Manual (DDM), Table 4-3.

Being located along the Port of Corpus Christi Inner Harbor and Corpus Christi Bay, sea level conditions were considered in the interior drainage assessment as well as potential wave overtopping along the Seawall. As stated in the scope of work, wave overtopping flows were determined for all storm events with and without allowances for future (50-year and 100-year) estimates of relative sea level rise (RSLR). Projected rates of RSLR were based on the U.S. Army Corps of Engineers (USACE) sea level rise calculator.

The interior drainage analyses performed for existing conditions are summarized in **Table 8**.

**Table 8 – Existing Conditions Scenarios**

| Scenario | Storm Event | | | |
|---|---|---|---|---|
| | 10% | 4% | 2% | 1% |
| Current Sea Level | ✓ | ✓ | ✓ | ✓ |
| 50-year RSLR | ✓ | ✓ | ✓ | ✓ |
| 100-year RSLR | - | - | - | ✓ |

In addition, analyses for the following alternative conditions were performed for the 1% ACE under current sea level conditions:

1. Failure of the Blucher Arroyo storm drain system and improvements;
2. Potential Hughes Street Pump Station (part of Downtown Drainage Improvements, Phase III, City Project 2226 A&B); and
3. Two improvement options based on design constraints which are discussed in more detail in subsequent sections.

## 5.2 Hydrology

In accordance with Chapter 4 of the DDM, the Natural Resources Conservation Service (NRCS) Curve Number loss method and Clark Unit Hydrograph method were used to develop runoff flow hydrographs. All hydrologic analyses were performed in XPSWMM. The hydrologic characteristics and methodology used in the model are discussed in subsequent sections of this memorandum. All geospatial data created as part of the hydrologic analysis are included in the geodatabase provided with this submittal.

## 5.2.1 Drainage Area Delineation

Drainage areas previously defined as part of the City's SWMP, Map K15, were used as the baseline delineation for this study. Drainage areas were further sub-divided using the 2011 1-foot LiDAR contours and existing drainage infrastructure GIS data. Drainage area inflow points, modeled as XPSWMM runoff nodes, were defined at major changes in storm drain sizes, groups of inlets (typically at intersections) storm drain outlets to channels, and major channel crossings.

Drainage areas maps are shown on **Exhibit B2** in **Attachment B**.

## 5.2.2 Precipitation Losses

The NRCS curve number (CN) methodology was used to compute runoff as outlined in the City's DDM. Composite CN's for each drainage area were calculated based on the existing conditions landuse dataset (**Exhibit B2**) and hydrologic soil group. Using the DDM Table 4-4, CN values were assigned to all landuse types based on the hydrologic soil group as shown in **Table 9**. Using the latest SSURGO soils data (2008) from the NRCS database, it was determined the entire study area is contained within Hydrologic Soil Group D soils.

**Table 9 – Curve Numbers**

| Landuse Type | Symbol | Imperv. % | Hydrologic Group | | | | Table 4-4 Description |
|---|---|---|---|---|---|---|---|
| | | | A | B | C | D | |
| Water | WATER | 100 | 100 | 100 | 100 | 100 | - |
| Agriculture | AG | 0 | 39 | 61 | 74 | 80 | *Open space/Parks (good cond.)* |
| Cons/Pres | CP | 0 | 39 | 61 | 74 | 80 | *Open space/Parks (poor cond.)* |
| Drainage Corridor | DC | 0 | 39 | 61 | 74 | 80 | *Open space/Parks (good cond.)* |
| Railroad ROW | ROWRR | 0 | 39 | 61 | 74 | 80 | *Open space/Parks (poor cond.)* |
| Vacant | VAC | 0 | 39 | 61 | 74 | 80 | *Open space/Parks (poor cond.)* |
| Estate Res. | ER | 20 | 51 | 68 | 79 | 84 | *Residential (1 ac)* |
| Low Density Res. | LDR | 25 | 54 | 70 | 80 | 85 | *Residential (1/2 ac)* |
| Medium Density Res. | MDR | 30 | 57 | 72 | 81 | 86 | *Residential (1/3 ac)* |
| High Density Res. | HDR | 65 | 77 | 85 | 90 | 92 | *Residential (1/8 ac)* |
| Mobile Homes | MH | 65 | 77 | 85 | 90 | 92 | - |
| Public/Semi-Public | PSP | 85 | 89 | 92 | 94 | 95 | *Commercial* |
| Park | PARK | 0 | 39 | 61 | 74 | 80 | *Open space/Parks (good cond.)* |
| Professional Office | PO | 85 | 89 | 92 | 94 | 95 | - |
| Commercial | COM | 85 | 89 | 92 | 94 | 95 | *Commercial* |
| Light Industrial | LI | 72 | 81 | 88 | 91 | 93 | *Industrial Districts* |
| High Industrial | HI | 85 | 89 | 92 | 94 | 95 | - |
| Streets | Road | 98 | 98 | 98 | 98 | 98 | *Paved Streets* |

*Curve Numbers reference City of Corpus Christi Drainage Design Manual Table 4-4.*

Calculated composite CN's for each drainage area and corresponding XPSWMM runoff nodes are summarized in **Table C1** in **Attachment C**.

### 5.2.3 Time of Concentration and Storage Coefficient

The Clark Unit Hydrograph, per DDM Section 4.2.4, was used to transform rainfall depths over the specified 24 hour period. Two parameters, time of concentration ($T_c$) and storage coefficient (R), were calculated to define the flow time and available storage characteristics for each drainage area. Time of concentration longest flow paths were developed for each drainage area, defining overland sheet flow, shallow-concentrated flow, and storm drain/channel segments accordingly. The DDM limits the allowable overland sheet flow segments to a range of 100 to 300 feet. With drainage areas less than 200 acres, the overland and shallow-concentrated flow components were calculated using the Upland Method (DDM Exhibit 4-1) which is dependent on slope and surface type coefficient of velocity, k-value. **Table 10** summaries the surface types and associated k-values used for this study. Similar to the City's SWMP, flow times through storm drains and culverts were calculated assuming velocities of 2 feet per second (fps) and 3 fps, respectively.

**Table 10 – Upland Method k-values**

| Landuse/Flow Regime | k-value |
|---|---|
| Forest or Meadow (Overland) | 2.5 |
| Fallow or Woodlawn (Overland) | 5.0 |
| Short Grass Prairie (Overland) | 7.0 |
| Cultivated (Overland) | 9.1 |
| Nearly Bare (Overland) | 10.3 |
| Grassed Waterway | 15.7 |
| Paved (Overland) & Gullies | 20.4 |

Additionally, Manning's equation was used to calculate open channel flow velocities, assuming a bank full condition with a maximum velocity of 6 fps. Velocity calculations were completed in Flowmaster and are included in the digital submittal. A minimum time of concentration of 10 minutes was assumed for all drainage areas as defined in the DDM for urban areas.

Drainage areas within the downtown area exhibit similar surface characteristics. Therefore times of concentration were calculated for select drainage areas at various areas to establish an area-time relationship. As shown in **Table C2** in **Attachment C**, drainage areas less than 10 acres had calculated times less than 10 minutes. For this study, all drainage areas located in the Downtown area (Power Street and Kinney Street Watersheds) with an area less than 10 acres used a 10 minute time of concentration. Times of concentration for all other drainage areas were calculated individually. Times of concentration calculations for each drainage area are summarized in **Table C3** in **Attachment C**.

Storage coefficients, R, were calculated using Equation 4-10 in the DDM which is based on the calculated times of concentration. As stated, R is calculated by applying a multiple of 3 to the time of concentration. The study area can be grouped into two major classifications, residential and urban/downtown. Within the downtown area, available storage is significantly different than in residential areas. To account for this potential storage differential, the multiplier was adjusted based on the landuse type as shown in **Table 11**.

Table 11 – Storage Coefficient Multiplier

| Landuse/Flow Regime | Multiplier |
|---|---|
| Commercial/Downtown | 2.0 |
| Residential/Mixed Use | 3.0 |

Storage coefficients for each drainage area are summarized in **Table C3** in **Attachment C**.

### 5.2.4 Rainfall

The hydrologic analysis used City DDM Table 4-3 rainfall depths for the storm frequency and duration previously discussed in Section 5.1. The total rainfall depths in inches for each storm event were distributed over the specified duration period using frequency storm hyetographs developed in HEC-HMS. To develop the nested storm hyetographs based on site specific depth-duration-frequency data, partial-duration rainfall depths were defined for the specified durations available in HEC-HMS. Rainfall depths referenced Table 4-3 of the DDM. Total rainfall depths used for this analysis are included in **Table 12**.

Table 12 - Total 24 Hour Rainfall Depths

| Frequency | | | |
|---|---|---|---|
| 10-Year | 25-Year | 50-Year | 100-Year |
| 7.12” | 8.94” | 10.48” | 12.18” |

Rainfall depths, frequency storm hyetograph, and HEC-HMS models are included in the digital submittal.

### 5.2.5 Local Peak Flows

Peak runoff flows for each drainage area are provided in **Table C4** in **Attachment C**. Generated runoff hydrographs were applied to specified “Runoff” nodes within the XPSWMM model. Being dynamic using full flow hydrographs, XPSWMM utilizes the St. Venant equations to hydraulically route the flow hydrographs through open channel and closed conduits. This method accounts for channel storage and attenuation in the closed conduits, and also takes into account the timing in the combining of flow hydrographs. **Exhibit B3.1** in **Attachment B** provides a general overall XPSWMM model layout.

## 5.3 Seawall Wave Runup and Overtopping

In addition to the runoff from local interior drainage areas, wave runup and overtopping also contributes flow, particularly along the Seawall at the entrances to the T-heads (at People’s Street and Lawrence Street) and L-head (at Cooper’s Alley). Overtopping flow per foot hydrographs for each storm event with and without allowances for future (50-year and 100-year) estimates of RSLR were developed over a 24 hour duration. These hydrographs were applied to each respective XPSWMM model via 2D flow areas to account for the additional overland flow contributing to the inundation within the Downtown area. See the Seawall Overflow Memorandum in **Attachment D** for details of the overtopping analysis. **Table 13** summarizes the developed wave overtopping peak flow rates for each Seawall subsection based on the defined segment lengths.

Table 13 – Wave Overtopping Peak Flow Rates (cfs)

| Seawall Subsection | Existing Modeling Transect | Segment Length (ft) | 10% ACE | | 4% ACE | | 2% ACE | | 1% ACE | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | (1) | (2) | (1) | (2) | (1) | (2) | (1) | (2) | (3) |
| SW1 | FEMA 35 | 1,467 | 1 | 5 | 4 | 17 | 9 | 43 | 54 | 246 | 1,781 |
| SW2 | New 1 | 674 | 0 | 1 | 0 | 3 | 1 | 8 | 10 | 53 | 484 |
| SW3 | New 2 | 1,749 | 1 | 5 | 4 | 17 | 9 | 43 | 55 | 251 | 1,831 |
| SW4 | New 3 | 3,914 | 0 | 2 | 2 | 10 | 5 | 30 | 40 | 237 | 2,427 |
| SW5 | FEMA 10 | 1,095 | 2 | 7 | 5 | 23 | 13 | 54 | 67 | 279 | 1,786 |
| SW6 | New 4 | 1,111 | 3 | 10 | 8 | 33 | 20 | 75 | 93 | 358 | 2,099 |

*(1) Current Sea Level*
*(2) 50-Year Regional Sea Level Rise*
*(3) 100-Year Regional Sea Level Rise*

## 5.4 Hydraulic Model Development

As with the hydrology, XPSWMM was used to perform the hydraulic analysis of all interior drainage conditions. The hydraulic analysis was performed in accordance with USACE, City DDM, City of Fort Worth (CoFW) Stormwater Criteria Manual (SWCM), Federal Highway Administration (FHWA), USACE HEC-RAS Reference, and XPSWMM Reference manuals. The following methods and data were used for the hydraulic components of the interior drainage analysis.

### 5.4.1 General Assumptions

Several assumptions were used in the development of the XPSWMM models. These assumptions were implemented to simplify certain components of the models which were not required for this analysis.

- All storm drain manhole and inlets were linked to the 2D surface and flow was allowed to transfer freely between the underground storm drain and surface components. Where system capacity remained available,1D/2D interface lines were added to ensure runoff was being captured by the system.
- Only storm sewer systems of significant size (24” and larger) were incorporated into the models unless system main trunks included smaller sizes.
- FHWA inlet types available in XPSWMM were applied to all culverts for inlet control evaluation. Inlet types were assigned based on field observations.
- Modeled storm sewer systems did not include all manholes, bends, and secondary lateral systems.
- 30-foot x 30-foot 2D grid cell provide sufficient surface detail for modeling purposes.
- All XPSWMM 1D and 2D elevations referenced the NAVD 88 vertical datum.
- The XPSWMM 2D component surface was developed using 2011 LiDAR DEM data.
- Where as-builts were not available, drainage system invert elevations and sizes were estimated using the GIS stormwater dataset provided by the City.

### 5.4.2 Hydraulic Losses

System losses throughout the hydraulic models were accounted for through defining loss coefficients for storm sewers (entrance, bends, manholes, and outlets), culverts, channel contraction/expansion and surface roughness. Loss coefficients used in the hydraulic models are summarized in **Table 14**. Loss coefficients reference the City DDM Tables 5-7 and 5-10, CoFW SWCM Table 3.12 and 3.14, FHWA Hydraulic Engineering Circular No. 22 Table 7-5, and USACE HEC-RAS Reference Manual, but were modified as needed for application in XPSWMM. Minor head loss coefficients were applied at the upstream (US) or downstream (DS) side of a hydraulic node (junction box, etc.) depending on the reference being used. XPSWMM applies expansion and contraction using one coefficient versus the common two coefficients used in HEC-RAS.

**Table 14 – XPSWMM Summary of Loss Coefficients**

| Reference | Minor Loss Description | Loss Coefficient | US Pipe | DS Pipe |
|---|---|---|---|---|
| City DDM, Tables 5-7 & 5-10 | Inlet on Main Line with Branch Lateral | 0.25 | ✓ | - |
| | Manhole on Main Line with 45° Lateral | 0.50 | ✓ | - |
| | Manhole on Main Line with 60° Lateral | 0.35 | ✓ | - |
| | Manhole on Main Line with 90° Lateral | 0.25 | ✓ | - |
| | Exit Loss | 1.00 | ✓ | - |
| | Headwall (Square Edge) Entrance Loss | 0.50 | - | ✓ |
| CoFW SWCM, Tables 3.12 & 3.14 | Inlet or Manhole at Beginning of Line | 1.25 | - | ✓ |
| | 90° Bend | 0.50 | - | ✓ |
| | 60° Bend | 0.43 | - | ✓ |
| | 45° Bend | 0.35 | - | ✓ |
| | 22.5° Bend | 0.20 | - | ✓ |
| | Sudden Enlargements (D2/D1 ≤ 1.2)* | 0.10 | ✓ | - |
| | Sudden Contractions (D2/D1 ≤ 1.2)* | 0.08 | - | ✓ |
| FHWA HEC-22, Table 7-5 | Access Hole - Straight Run | 0.15 | - | ✓ |
| | Access Hole - Angled Through - 90° | 1.00 | - | ✓ |
| | Access Hole - Angled Through - 120° | 0.85 | - | ✓ |
| | Access Hole - Angled Through - 135° | 0.75 | - | ✓ |
| | Access Hole - Angled Through - 157.5° | 0.45 | - | ✓ |
| HEC-RAS Manual | Channel Expansion/Contraction Loss - Gradual | 0.1 (Uniform/Natural Section) | - | - |
| | Channel Expansion/Contraction Loss – Abrupt | 0.4 (Bridges) | - | - |

**D2/D1 is the ratio of the larger to smaller pipe diameter at the transition*

Standard roughness values were used where applicable for the storm drains, culverts and channel sections. Roughness values assumed in this study are summarized in **Table 15**.

**Table 15 – Summary of Manning's Roughness Coefficients**

| Description | Manning's n-value |
|---|---|
| High –Density Polyethylene (HDPE) Pipe | 0.012 |
| Precast Reinforced Concrete Pipe/Box | 0.013 |
| Concrete-Lined Channel | 0.015 |
| Grass Lined Channel (regular maintenance) | 0.030 |

## 5.4.3 XPSWMM Components

The drainage infrastructure within the modeling limits included 1D channel, storm drain, and culvert drainage components along with 2D overland drainage components. Two pump stations, at Outlet A, were incorporated using the multi-link pump component. An overall layout schematic of the XPSWMM model is provided on **Exhibit B3.1** in **Attachment B**. A more detailed layout schematic of the Downtown area is shown in **Exhibit B3.2**.

***1D Components***

Single 1D links were used to define single storm drain and culvert conduits. Multiple pipe or box culvert conduits of identical size remained as a single link but the number of conduits was specified in the “Conduit Factors” component within the XPSWMM interface. Multi-links were also used to model multiple box culverts of difference sizes or box culvert and channel sections in combination. This method allows for conduit specific properties to be selected including size, roughness, and hydraulic losses.

With a 30-foot x 30-foot 2D grid cell size, modeling the existing channels as 2D components would insufficiently represent the available channel capacity. Therefore, a 1D “natural section” link, cut using the LiDAR DEM supplemented with as-built plans, was used to define channel sections. Contraction/expansion, entrance and exit losses were applied at the upstream and downstream ends of each conduit within the “Conduit Factors” component.

Existing pump rating curves (**Table 3**, **Table 4**, and **Table 6**) and pump operations (**Table 5** and **Provided** pump operations for the Power Street pump station were modified and included to hydraulically simulate pump effectiveness within the hydraulic model. **Table 7** summarizes the pump operations for the Power Street pump station. Based on the operations, an initial depth of 3.92’ was assumed within the system. With a wet well invert of -10.83’ NAVD 88, this equated to an initial elevation of -6.91’ NAVD 88.

Table 7 7) were incorporated into the model using the pump option of a multi-link. Multi-links allow the user to apply conduit, pump, weir, or user rating curve data to the same link segment. In this case, the multi-link included the three and six pump configurations for the Power Street and Kinney Street pump stations, respectively. Additionally, the full storage volume available within the pump house system was applied to the system nodes within the XPSWMM model. The available storage for the Power Street pump station ranged from the invert at El. -10.83 feet NAVD 88 to the top of the chamber at El. 8.38 feet NAVD 88 as shown on Sheet 4173 of the 1947 As-builts. For the Kinney Street pump station, available storage ranged from the invert at El. -18.5 feet NAVD 88 to the top of the chamber at El. 20 feet NAVD 88 as shown on Sheet 4 of the 2008 pump station plans.

All storm drain manhole and inlet XPSWMM nodes within the system, except system outlets, were hydraulically connected to the 2D surface using the “Link Spill Crest to 2D” or “Link Invert to 2D” options. Spill crest and invert elevations were set equal to the equivalent surface elevation. Inlet capacities were assumed to not restrict flow and therefore were not applied. The 2D grid elevations automatically adjusted to match the corresponding node crest/invert elevations as those were taken directly from the LiDAR surface. The LiDAR dataset did not include the deck elevations at all channel crossings. Therefore, the decks were modeled in the 1D component as weirs. Weirs were set at the crown of the streets and weir coefficients were specified accordingly. In XPSWMM, deck thickness is not physically specified in the bridge parameters. To account for a “pressure flow” condition, a maximum height within the natural channel section of the bridge was specified equal to the difference in elevation between the channel flow line and the low chord elevation of the structure as shown in provided as-builts and design plans. This limits the active flow area until the WSEL exceeds the set weir elevation, in which case active flow will spill. Channel segments modeled as 1D components were linked to the 2D surface using the “Link Spill

Crest to 2D" option with the spill crest set to the channel bank or cross section end point elevation. Inactive areas along the 1D channel areas were applied to ensure there was no double counting of storage between the 1D and 2D simulations.

### *2D Components*

To accurately account for hydraulic losses on the 2D surface, overland flow Manning's n-values were applied for the different land uses within the study area. Using the current City landuse dataset, a modified 2D landuse was created to make minor changes and reclassify land uses to be more applicable for the model to appropriately simulate complex overland flow characteristics. **Table 16** summarizes n-values used to define 2D surface roughness.

**Table 16 – 2D Landuse Manning's Roughness Coefficients**

| Description | Manning's n-value | Description | Manning's n-value |
|---|---|---|---|
| Open Space | 0.03 | Commercial | 0.02 |
| Undeveloped | 0.04 | Light Industrial | 0.025 |
| Residential (All Types) | 0.035 | Heavy Industrial | 0.04 |
| Railroad Row | 0.04 | Professional Office | 0.02 |
| Mobile Homes | 0.025 | Streets | 0.02 |
| Public Area | 0.02 | | |

In addition to applying a surface roughness to the 2D component of the hydraulic models, building planimetric data was used to block flow through the building areas by designating them as "inactive areas".

### *System Boundary Conditions*

System boundary conditions for the hydraulic model were used to define tailwater elevations at all storm drain and channel outlets, define initial depths at the start of the model simulation, and account for wave overtopping flows developed as part of the wave overtopping analysis for the Seawall (see Section 5.3).

As discussed in the Wave Overtopping Memo in **Attachment D**, time series stage hydrographs were generated over a 24 hour period for the twelve scenarios included in this study. Stage hydrograph distributions began with the mean sea level during normal conditions **(Table 17**) with peaks set equal to the stillwater storm surge WSELs as defined in the FEMA preliminary Flood Insurance Study (FIS) for Nueces County Table 6 dated October 23, 2015 and summarized in Table 2 of the Wave Overtopping Memo.

**Table 17 – Normal Sea Level Conditions**

| Scenario | Storm Event | | | |
|---|---|---|---|---|
| | 10% | 4% | 2% | 1% |
| Current (2016) Sea Level | 1.141 | 1.392 | 1.575 | 1.917 |
| 50-year RSLR | 1.438 | 1.689 | 1.871 | 2.214 |
| 100-year RSLR | N/A | N/A | N/A | 2.601 |

The stage hydrographs were applied to all storm drain outlets as shown in **Exhibit B4** in **Attachment B**. Additionally, 2D head boundaries along the Seawall and in the Salt Flats area were incorporated into the model to apply the same stage hydrograph boundary conditions along the 2D grid. This was performed to simulate actual tailwater conditions during a storm event which will accurately influence upstream overland 2D flows. A third 2D head boundary was applied along the west 2D grid boundary within the Salt Flats industrial area. This boundary used an arbitrary elevation set below the actual ground surface elevation to simulate a “free flow” condition which allows 2D overland flow to run off the grid if needed. Without this boundary condition set, the 2D grid edge acts as a wall causing artificial inundation buildup.

All storm drains discharging into the Corpus Christi Bay have flap gates, preventing backwater (negative flow) from entering the drainage systems. Driven by WSEL head, storm drains can only discharge into the Bay once the hydraulic grade elevation within the system is greater than the outlet boundary condition (sea level). Therefore, the minimum hydraulic grade elevation within the storm drain systems is considered equal to the sea level elevation during normal conditions. To account for this condition, all outfalling storm drain segments were set to a “downhill only” flow condition to simulate the flap gates and correlating initial depths were applied to the appropriate nodes to simulate initial sea level elevations (**Table 17**). **Exhibit B4** illustrates the drainage systems considered under this condition. Similar to the storm drain systems, the canals east of the Salt Flats levee within the Port of Corpus Christi – Southside were controlled by the outlet conduit to the Salt Flats channel north of the railroad, near Brewster Street as shown in **Exhibit B4**. Additionally, the Salt Flats area channel applied a similar initial depth condition. There are no control structures at the channel outlet to the Inner Harbor; therefore, it was modeled with a “free flow” flow condition, allowing for positive or negative flow depending on upstream runoff and downstream tailwater conditions.

Three unique internal boundary conditions at the Power Street pump station, Kinney Street pump station, and 36” RCP lateral (L10.01 and L10.02a) along Kinney Street, from Water Street to the Seawall, were incorporated into the hydraulic model. Interior storm drain systems discharging to the Power Street and Kinney Street pump stations are influenced by pump operations. As discussed in Section 4.5.2, the initial depths at the two pump stations were 3.92’ and 0.00’, respectively. Therefore, initial depths within the hydraulically connected upstream storm drain nodes were set accordingly. Since the Kinney Street pump station empties the wet well, the upstream storm drain system was assumed to be dry initially. The 36” RCP lateral along Kinney Street originally extended west of Water Street as shown in the Bay Front Protection as-builts dated 1938. When the 8’x5’ box along Water Street was constructed in 2007, the existing 36” RCP was plugged west of Water Street and connected to the box at an invert of -2.3 feet NAVD 88. The next downstream manhole invert of the 36” RCP lateral is higher (-0.87 feet NAVD 88), causing a high point. This high point produces a permanent pool in the storm drain east to the outlet at the Seawall. The equivalent initial depths were applied at the appropriate storm drain nodes to simulate this permanent pool elevation. See **Exhibit B4** for depiction of unique boundary conditions.

As discussed in Section 5.3, wave overtopping flow hydrographs were developed as part of the Wave Overtopping analysis. These flow hydrographs were applied to the hydraulic model using the 2D flow area component distributed evenly over the area. Flows developed for each segment of the Seawall were further divided to ensure active application to the 2D surface across the entire length of the Seawall. A raised median along the majority of S. Shoreline Boulevard influences drainage patterns of wave overtopping flow from the Seawall. With a priority objective of assessing interior system capacity in the Downtown area, the 2D flow areas (**Exhibit B4**) were applied at the median high point to allow full flow contributing Downtown. Although there will be a measureable volume of storage along the northbound S. Shoreline Boulevard lanes due to containment by the raised median and elevated Seawall, the LiDAR surface and 2D grid does not provide the necessary detail for modeling. Therefore, this possible storage was not considered for this study.

***Topography Components***

As previously stated, standard 30-foot x 30-foot 2D grid cells were used to define the 2D surface within the XPSWMM model. A single elevation representing each grid cell referenced the 2011 LiDAR DEM which was used to develop an xptin file within the XPSWMM software. Although the established grid cell size was sufficient for modeling the surface hydraulics within the study area, small topographic details in select areas were not accurately characterized. However, XPSWMM allows the user to incorporate surface modification components to alter the generated surface by adding more detail. To refine these areas of concern, breakline "ridge" components were incorporated to further define the modeling surface. Each of the refined areas is discussed in more detail in the subsequent sections and can be viewed on **Exhibit B5** in **Attachment B**.

> ***Hughes Street**: Due to the proximity of buildings along Hughes Street near N. Shoreline Boulevard, the roadway elevation data lacked the level of detail needed to properly simulate street flow conveyed along Hughes Street westward to Water Street. The added ridge lines removed artificial roadway highs to allow for positive drainage.*
>
> ***N. Shoreline Boulevard Median**: A prominent median runs along N. Shoreline Boulevard from the American Bank Center north of Hughes Street south to John Sartain Street at which structures exists, providing a more gradual high point. This continues southward to just past the Furman Avenue intersection. Ridge lines were included to ensure that the natural high point influenced the 2D hydraulic calculations. As mentioned in the above section, the 2D flow areas representing wave overtopping flows were applied along this ridge line.*
>
> ***Top of Seawall**: Based on as-builts and field observations, the top of Seawall elevations are approximately two feet higher than the adjacent street elevations. With a LiDAR DEM containing an approximately 11-foot x 11-foot raster cell size, the wall detail was present. Ridge lines were placed along the Seawall with top of wall elevations correlating to those used in the wave overtopping analysis. Those top of wall elevations can be referenced in Table 3 of the Wave Overtopping Memo.*
>
> ***Salt Flats Levee**: One of the key assumptions in evaluating the interior drainage system of Downtown Corpus Christi is that the levee gates are operational and functioning during a storm event. However, the LiDAR surface includes the road elevations at each gate location. To simulate this assumption, a ridge line was added to artificially raise the surface to the equivalent height of the Salt Flats Levee wall. Ridge lines along the top of levee wall were also incorporated to ensure that top of wall elevations were represented correctly in the hydraulic model.*
>
> ***Museum Floodwall**: Similar to the previous location, the surface was modified to simulate the floodwall at the Museum of Science & History (**Figure 4**) with gate raised sufficiently to block surge elevations from Corpus Christi Bay.*

Additional XPSWMM model information can be found in the Background Information document included with the XPSWMM models in the digital submittal. All data used to build the XPSWMM model are included in a geodatabase provided with this submittal.

# 6 Floodplain Inundation Mapping

Maximum inundation limits were mapped in XPSWMM using the LiDAR DEM and computed model maximum WSEL and depth outputs at each grid cell. Developed output grid contours were exported to GIS in raster and vector format. The raster output datasets were used to display overall mapping limits within the entire study area. For the regulatory mapping areas of concern, Salt Flats area and Downtown (Seclusion Zone), output contour datasets influenced the delineation of floodplain boundaries. The SFHA was defined as a Zone AE floodplain with static water level BFEs. Similar to the mapping approach defined in the FEMA LAMP study, shallow sheet flow due to overtopping along the Seawall was delineated as a Zone AO floodplain with static depths.

To delineate the static elevation Zone AE boundaries and static depth Zone AO boundaries within the Downtown area, maximum extents output developed coarse floodplain boundaries. Refinements were made to remove small ambiguous voids and smooth the boundary limits. As previously discussed, 2D flow was allowed to flow around buildings resulting in large floodplain voids. For mapping purposes, voids of buildings considered “flooded” were filled. Buildings were considered “flooded” or “at-rick” if adjacent flood depths exceeded 0.5’. In some instances, flooding remained within the street areas accounting for shallow depth flooding. In accordance with FEMA mapping guidance, all Zone AO flow depths less than 1.5 feet were set to 1 foot. Coarse floodplain delineations resulted in disconnected floodplain areas. When appropriate, floodplain boundaries were modified to unify street flooding delineation areas. WSEL contour output datasets were used to define common static elevation zone (rounded to nearest foot) or sheet flow depth boundaries.

Results mapping is discussed in more detail in the subsequent section. Raster output results for all existing conditions scenarios simulated are included in the digital submittal.

# 7 Existing Conditions Results

The developed XPSWMM models for existing conditions[3] were executed for the four storm events under the three scenarios summarized in **Table 8**, resulting in a total of nine simulations. Maximum WSELs and flows for the 10%, 4%, 2%, and 1% ACE under current (2016) sea level conditions are summarized for each major drainage system in **Table C5a** and **Table C5b** in **Attachment C,** and approximate flood inundation limits of the entire study area for the 10% and 1% ACE are shown in **Exhibit B6** in **Attachment B**.

A preliminary assessment of inundation depth results within the Seclusion Zone was completed to determine overall flood impacts for each storm event. Referencing **Figure 6** to **Figure 9**, the level of flood protection (maximum flood depth, floodplain acreages, and approximate number of buildings at risk within each identified zone) for each storm event was summarized (**Table 18)** for specified zones within the Seclusion Zone. Note that the maximum flood depths occur in relatively isolated locations as designated by the green circles in the figures. The average flood depths are generally less than 1 foot for the 10%, 4%, and 2% ACE conditions.

[3] Existing conditions were modeled assuming no breaching of the Salt Flats Levee or other elements of the City’s perimeter flood protection system.

**Table 18 – Level of Flood Protection (Downtown Area)**

| Storm Event (ACE) | Zone 1 | | | Zone 2 | | | Zone 3 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Max Flood Depth[1] | Floodplain Acreage[2] | Approx Bldgs at Risk[3] | Max Flood Depth[1] | Floodplain Acreage[2] | Approx Bldgs at Risk[3] | Max Flood Depth[1] | Floodplain Acreage[2] | Approx Bldgs at Risk[3] |
| 10% | 5.1’ | 43.31 (40%) | 31 | 1.8’ | 30.15 (89%) | 3 | 1.1’ | 29.17 (95%) | 6 |
| 4% | 5.3’ | 48.14 (36%) | 37 | 2.1’ | 42.06 (74%) | 20 | 1.3’ | 38.50 (93%) | 9 |
| 2% | 5.3’ | 50.39 (33%) | 37 | 3.8’ | 61.38 (65%) | 40 | 1.6’ | 44.55 (90%) | 9 |
| 1% | 5.4’ | 52.92 (30%) | 38 | 4.8 | 73.44 (52%) | 52 | 1.8’ | 52.14 (87%) | 10 |

*[1]Maximum flood depths not contained within Drainage Corridor (channels, roadside ditches). Locations identified in referenced Figures.*
*[2]Area of floodplain with depths less than 0.5’ provided as percentage.*
*[3]A total of 356 buildings considered for at-risk assessment. Buildings adjacent to depths greater than 0.5’ are considered at-risk.*

For all storm events, the worst areas of flooding occurred in the northeast portion of Downtown along Hughes Street (Zone 2) and within the Port Southside industrial area (Zone 1) from Hwy 181 to the Salt Flats Levee. This is due to insufficient capacity of the existing storm drain systems along Hughes Street and elevated tailwater conditions in the downstream system along Water Street which is controlled by the Power Street pump station. Flooding within the Port Southside area can be attributed to the inability to drain until capacity is available within the Salt Flats channel since a backflow prevention system is in place at the Salt Flats Levee. Wave overtopping flows along the Seawall added shallow depth flooding along Shoreline Boulevard and down side streets to Water Street but did not have a major impact (average 1% ACE WSEL increases of less than 2 inches) on overall flooding within the Downtown area. In comparing the contributing runoff volumes produced by the local rainfall runoff and wave overtopping, overtopping accounted for much less volume as shown in **Table 19**.

**Table 19 – 1% ACE Flood Contribution Volumes**

| Watershed | Local Rainfall Runoff (ac-ft) | Wave Overtopping (ac-ft) | Total (ac-ft) |
|---|---|---|---|
| Power Street | 287 (90.0%) | 32 (10.0%) | **319** |
| Kinney Street | 66 (91.0%) | 6 (9.0%) | **72** |
| Coliseum | 77 (76.4%) | 24 (23.6%) | **101** |
| Furman | 47 (82.7%) | 10 (17.3%) | **57** |

The refined 1% ACE floodplain limits depicting flood zones and base flood elevations (BFEs) within the Salt Flats and Downtown areas is included in **Exhibit B7**. Study and mapping limits have been identified on the exhibit.

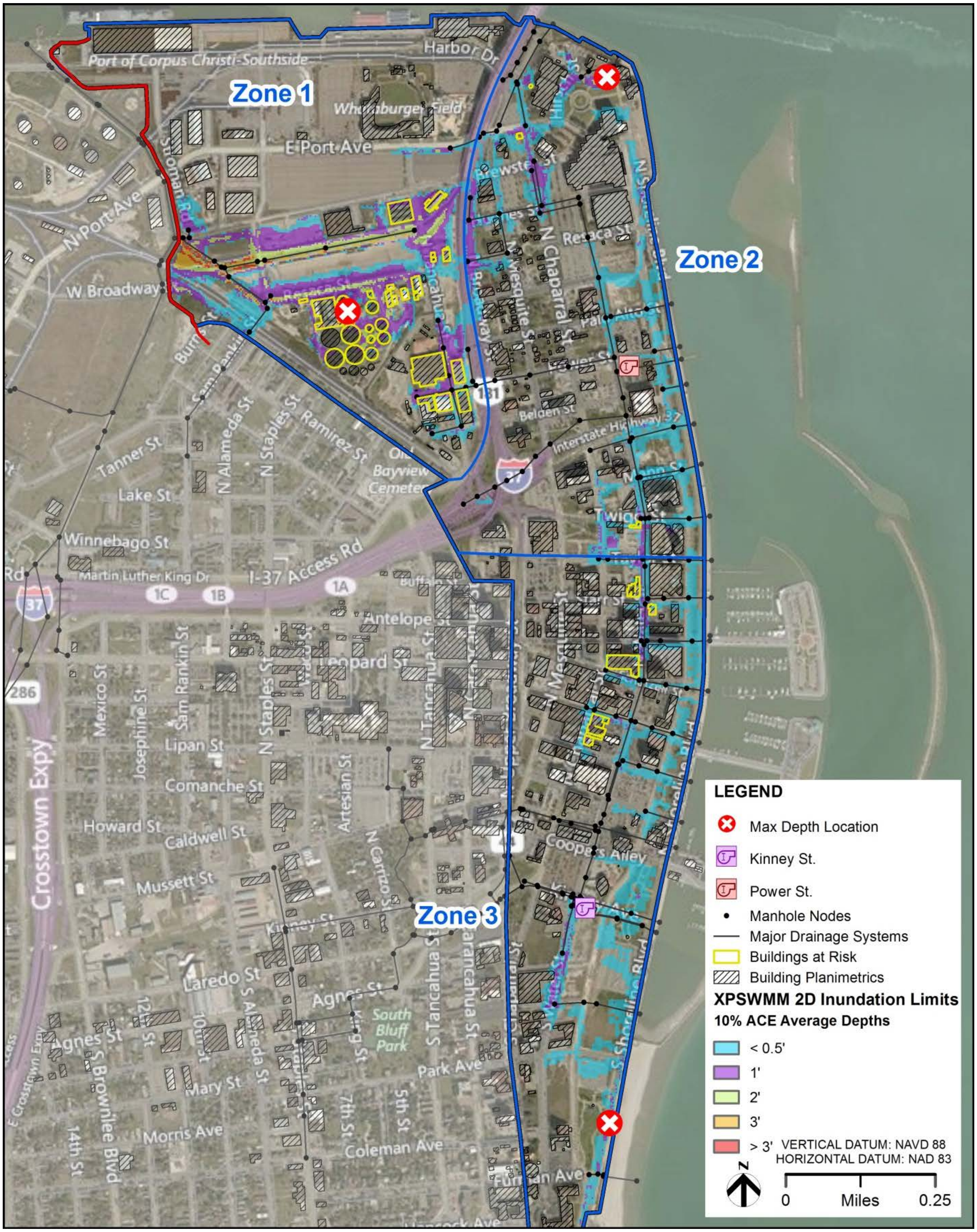


**Figure 6 – 10% ACE Inundation Depth Limits (Downtown)**

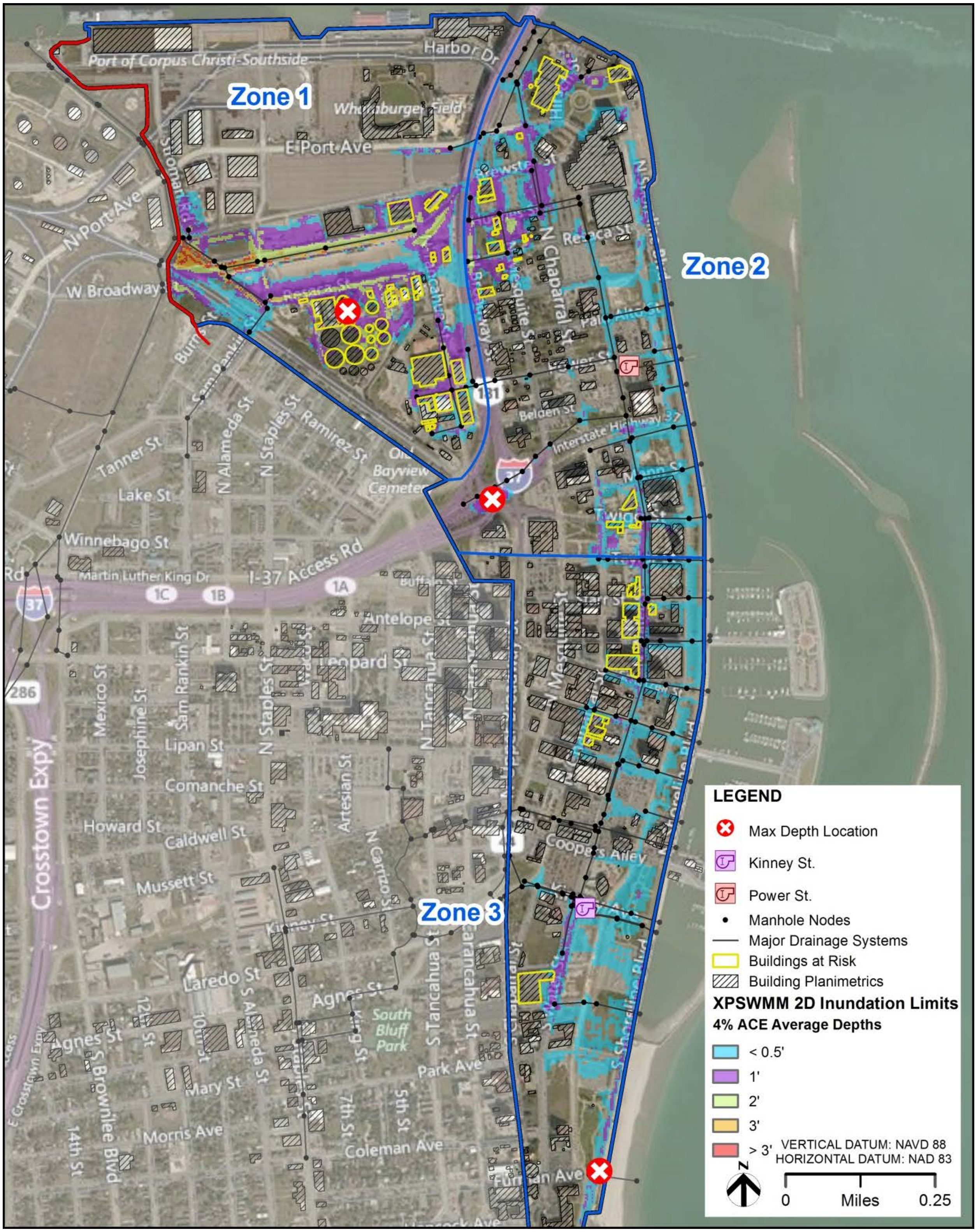


**Figure 7 – 4% ACE Inundation Depth Limits (Downtown)**

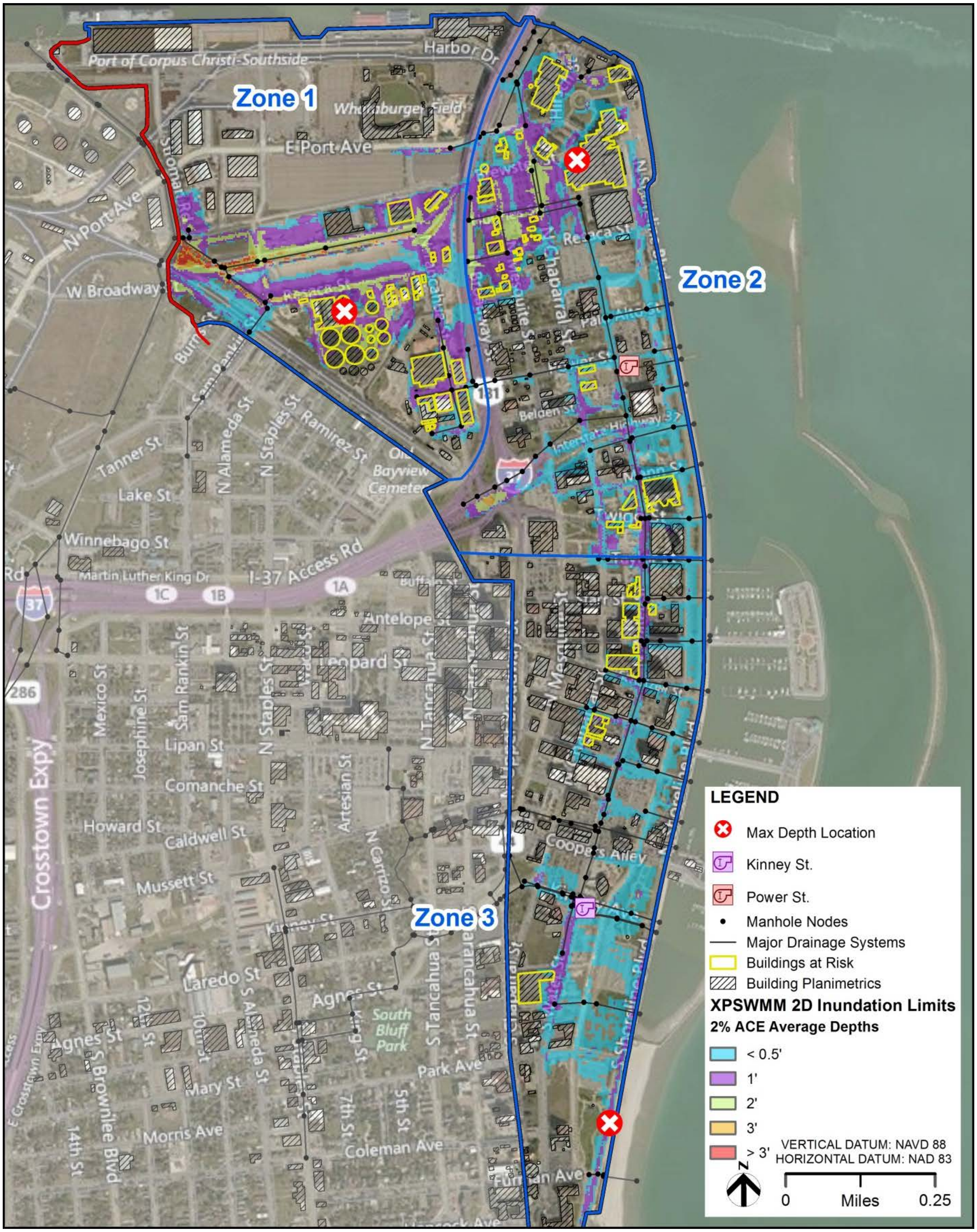


**Figure 8 – 2% ACE Inundation Depth Limits (Downtown)**

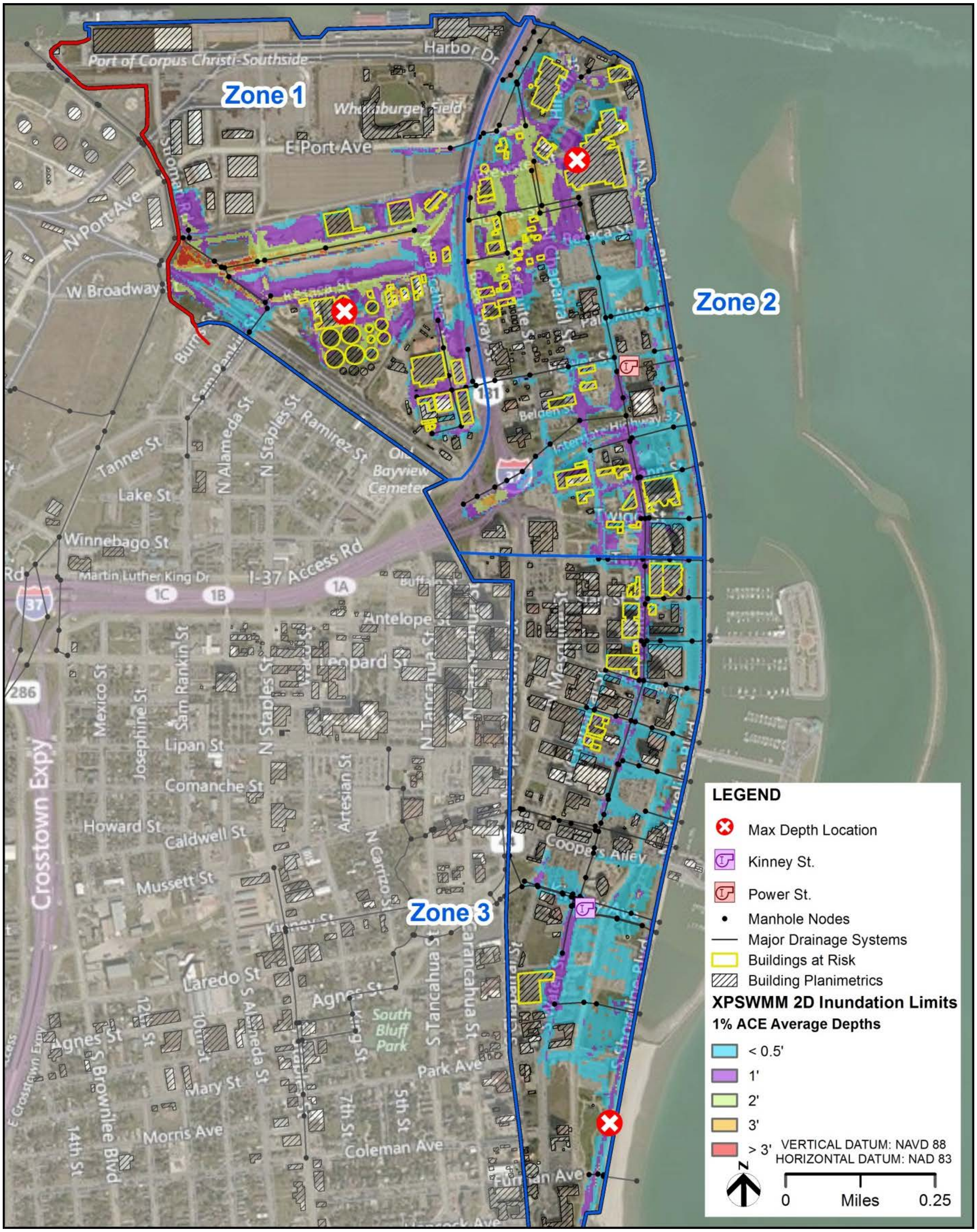


**Figure 9 – 1% ACE Inundation Depth Limits (Downtown)**

## 7.1 Comparison to FEMA Results

A comparison of the 1% ACE floodplain mapping extents to the FEMA FIRM Panel 48355C0320G with a preliminary Effective Date of October 23, 2015 was developed by comparing the refined 1% ACE floodplain and BFEs within the Salt Flats area (Turning Basin Tributary) to those depicted on the FEMA FIRM Panel. In the XPSWMM model, WSELs were generally lower in the Salt Flats area as shown on **Exhibit B8** in **Attachment B**. This can be attributed to differences in the hydraulic modeling approach between the two floodplain analyses. In using a fully dynamic 1D/2D hydrology and hydraulic model for this study, peak timing, downstream boundary condition, and channel routing greatly influenced resulting WSELs within the Salt Flats area. As stated in the methodology for downstream boundary conditions and local rainfall, the tailwater stage hydrograph was developed with a peak stage (surge elevation) occurring at 50% of the storm duration (24 hours). However, as defined in the City DDM, local rainfall produced flow hydrographs with peak flows occurring at 66% of the storm duration. As a result of the non peak-on-peak condition, WSELs within the Salt Flats area were generally controlled by the tailwater elevations with the local rainfall peaks occurring later in the storm event producing a second, lower peak WSEL (**Figure 10**). This modeling approach provides a hydraulic result similar to a typical storm event condition for this area.

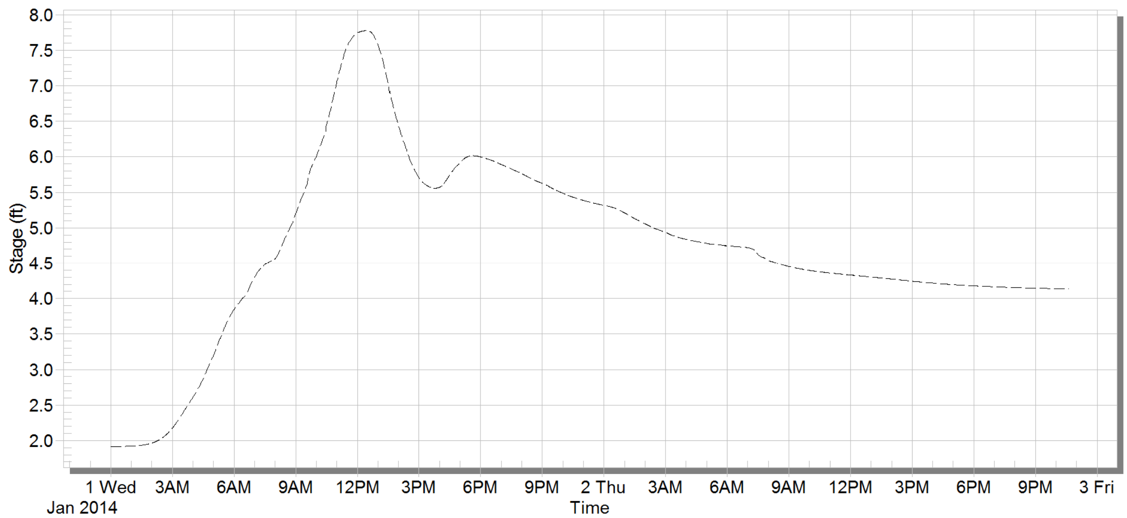


**Figure 10 – Salt Flats Area Stage Hydrograph**

Concurrent to this study, FEMA has been in the process of finalizing their LAMP plan for Nueces County previously developed in September 2015. As part of their study, a FLO-2D model was used to evaluate the extent of flooding within the Downtown area due to storm surge and local rainfall. Four scenarios were modeled to evaluate the flood protection system, as discussed in the report. Comparisons of the 1% ACE mapping extents from the XPSWMM model to the LAMP plan results, including wave overtopping combined with interior drainage and pump station operations, are shown on **Exhibit B9.1** and **Exhibit B9.2** in **Attachment B**. Mapping extents in the XPSWMM model were significantly different than the FLO-2D results within the Seclusion Zone. For example, in the XPSWMM model, areas south of Power Street experienced shallow ponding with depths from 1 foot to 3 feet, which is much lower than the WSEL of 7 feet (depths up to 4 feet) and 8 feet (depths up to 5.5 feet) NAVD 88 defined in the LAMP study. Major changes in floodplain extents between the studies can be attributed to the different modeling approach (e.g., the XPSWMM model better incorporated major storm drain systems and utilized more accurate pump characteristics). A side by side comparison is provided in **Figure 11** and **Figure 12**.

Based on the FLO-2D model, the Zone D[4] area (**Figure 12**) was originally mapped to WSEL 10 feet NAVD 88 established by the "natural valley[5]" approach as stated in the LAMP study, creating a composite floodplain designation of the different FEMA reach scenarios. Concurrent to this study, the City has tasked HDR with identifying and implementing improvements to the Salt Flats Levee system. The intent of these improvements is to reclassify the non-accredited levee system to a "freeboard deficient" designation, meeting all levee requirements established in Title 44 of the Code Federal Regulations Section 65.10 except the required 3' freeboard above base flood WSELs for riverine levee systems. Still classified as a non-accredited levee system, a system-wide Zone D area developed by the "natural valley" approach is still required under FEMA guidelines. However, as previously stated, this zone designation reflects an area of approximate, undetermined flood risk.

With the development of the more detailed XPSWMM in this study, flood risk is no longer undetermined. Since a "natural valley" approach model was not developed as part of this study, the Zone D area extents developed by the natural valley approach in the LAMP study were left unchanged but reclassified to a Shaded Zone X as shown in **Figure 11**, representing areas protected by levees from 1% annual chance flood. The zone designation change is desired based on this detailed study, supporting a more accurate representation of flood risk in the Downtown seclusion area.

[4] Zone D refers to areas in which flood hazard are undetermined, but possible.
[5] The natural valley approach assumes the Salt Flats Levee has been breached.

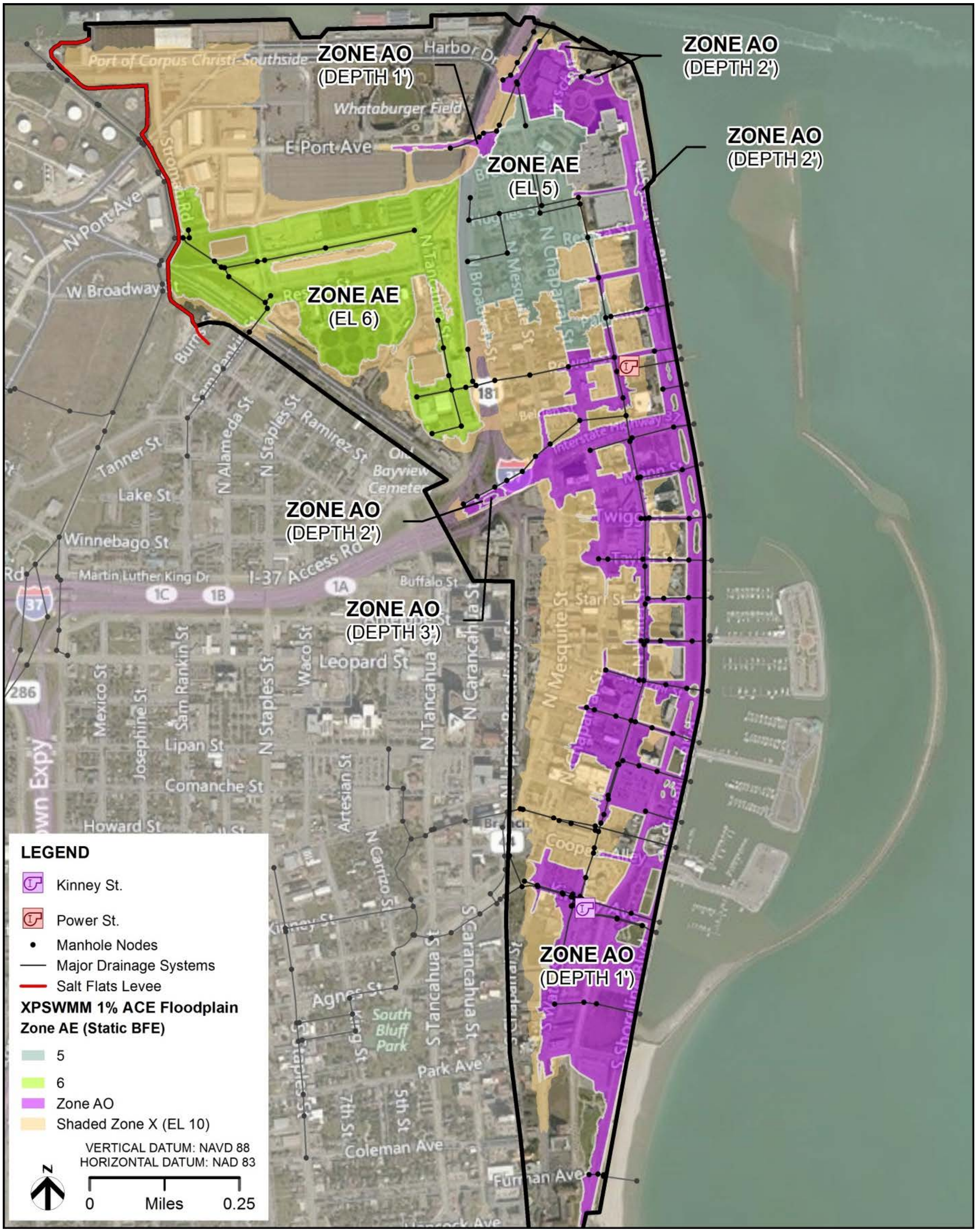


**Figure 11 – XPSWMM 1% ACE Floodplain Limits**

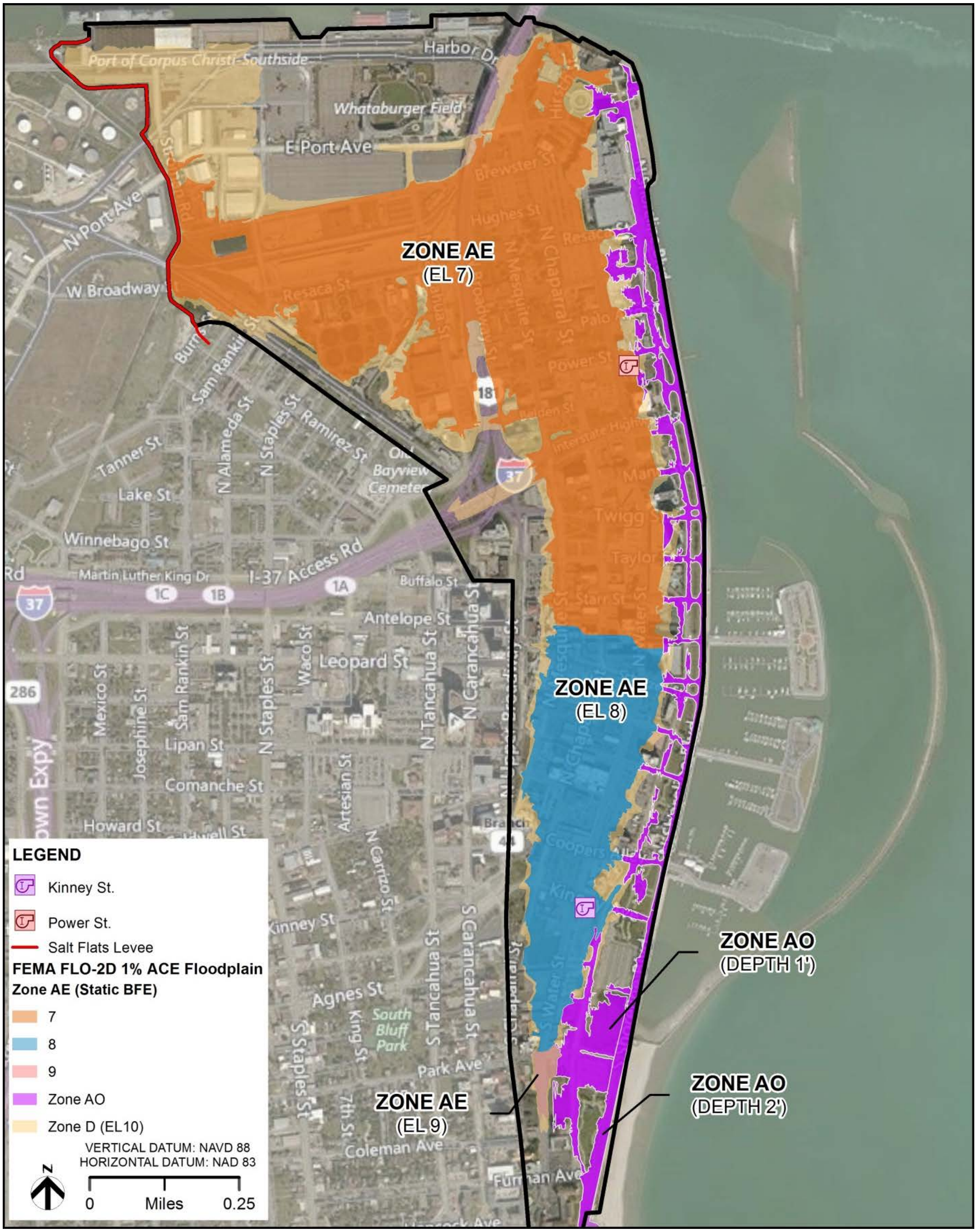


**Figure 12 – FEMA FLO-2D 1% ACE Floodplain Limits (Composite)**

## 8 Review Workshop

A three-hour workshop to discuss the 1D/2D XPSWMM hydraulic model of the Downtown Flood Protection System and inundation areas was held at the City's office on August 10, 2016. The meeting minutes are in **Attachment E** to this memorandum. A 1% ACE inundation exhibit (**Exhibit F1** in **Attachment F**) with all buildings displayed was developed for the workshop to discuss building selection for survey. The City selected the twenty (20) buildings (or in some cases representative groups of buildings) to receive finished floor surveys. They are listed in **Table 20**. The survey results for each selected building are included in **Attachment F** and were used to help characterize the risk of flooding at these buildings under the various storm scenarios simulated with the XPSWMM model.

**Table 20 – Surveyed Buildings**

| Bldg No. | Description | Bldg No. | Description |
|---|---|---|---|
| 1 | Bay Yacht Club | 11 | Concrete Street Amphitheater |
| 2 | U&I Restaurant | 12 | Braslau's Fine Home Furnishing |
| 3 | IBC Bank | 13 | Port of Corpus Christi |
| 4 | CC Municipal Court & Police Dept. | 14 | CC Wastewater Treatment Plant |
| 5 | Education Service Center Region 2 | 15 | Heritage Park - Galvan House |
| 6 | Several Retail Stores & Restaurants | 16 | Heritage Park - Sidbury House |
| 7 | The Cosmopolitan of Corpus Christi | 17 | Institution of Cultural Hispanica |
| 8 | Downtown Grill Steaks & Seafood | 18 | Texas State Museum of Asian Culture |
| 9 | CC Caller Times | 19 | Harbor Playhouse |
| 10 | CC Fire Station | 20 | CC Hispanic Chamber of Commerce |

Based on the building selection and priority areas set by the City, sump constraints were defined to remove buildings from being "at risk" (inundation depths greater than 0.5') and used to establish the necessary improvement options 1 and 2 based on the specified buildings of interest for each scenario.

## 9 Pump System Alternatives Analysis

Using the developed existing conditions hydrologic and hydraulic model incorporating the major dynamics of the Downtown Flood Protection system (floodwalls, wave overtopping, pump stations, and interior drainage systems), a more detailed inundation area was established, identifying areas susceptible to flooding. As part of this study, the City requested that HDR evaluate one potential existing system failure and several proposed project improvements for the 1% ACE storm at current sea level conditions. As stated in Section 5.1, alternatives were modeled to evaluate a theoretical failure of the Blucher Arroyo system, the completed design of the Hughes Street pump station, and three potential project options to remove selected building from being "at risk." A detailed discussion of each alternative and findings are provided in the following sections.

### 9.1 Blucher Arroyo Pressure Box Failure

During a storm in October 2002, the Blucher Arroyo storm drain system experienced a system failure along N. Mesquite Street approximately 280 feet south of John Sartain Street. A breach in the pressure box system caused a street collapse approximately 10 feet wide allowing a significant amount of flow to surcharge onto the surface. As part of the emergency response team, HDR developed reconstruction plans to repair the box.

Currently, the Blucher Arroyo system is sealed and modeled under pressure from N. Upper Broadway Street to the outlet at the Seawall. To model a system failure and identify the necessarily mitigation improvements, the 12' x 4.75' pressure box within N. Mesquite Street was modified to allow surcharging, simulating a theoretical system breach as shown in **Layout G1** in **Attachment G**. The hydraulic model was executed and resulting impacts were documented. Due to the failure, the Blucher Arroyo system surcharged to Mesquite Street for approximately 70 minutes with an estimated peak of 85 cfs. Based on modeling of this failure, flooding occurred with maximum depths greater than 1.0' along John Sartain Street from Mesquite Street to Water Street as shown in **Exhibit B10** in **Attachment B**. WSEL increases shown in the northern portion of Downtown (Hughes Street area) were insignificant (less than 0.05' change in depth). Based on the City's current drainage system, maximum WSELs with and without a hypothetical pressure box failure were compared and summarized for each major drainage system in **Table C6** in **Attachment C**.

In order to mitigate the inundation that would be caused by another failure of the pressure box system, the following improvements were identified for the Kinney Street system. See **Layout G1** for schematic of proposed improvements.

- Upsize 15" to 18" storm drain system along John Sartain Street and Chaparral Street to 4'x4' storm drain box.
- Tie upsized lateral system along John Sartain Street to existing 8'x5' storm drain box within Water Street.
- Extend 4'x4' storm drain box through parking area south of Mamma Mia's Restaurant from Chaparral Street to Mesquite Street.
- Improvements assume adequate inlet capacity provided to capture surface runoff.

The system failure hydraulic model was modified to include these proposed system improvements and executed. Maximum WSELs with these improvements were compared to existing conditions and summarized for each major drainage system in **Table C6** in **Attachment C**. Approximate flood inundation limits of the entire study area are shown in **Exhibit B11** in **Attachment B**.

With these proposed improvements, flooding caused by the system breach was reduced to depths less than 0.5' within the local area. The proposed improvements were shown to provide flood protection for buildings within the area if another system breach were to occur. The increased storm drain capacity of the John Sartain Street system was shown to increase inundation depths by a maximum of 0.3' within Water Street near Williams Street with negligible depth increases of less than 0.1' near the Kinney Street Pump Station. These downstream system impacts can be attributed to the additional runoff volume captured by the proposed improvements.

## 9.2 Proposed Hughes Street Pump Station

Urban Engineering has completed the design of a third pump station, which would be located at Hughes Street, and associated improvements, referred to as the Downtown Drainage Improvements, Phase III, City project 2226 A&B. For the current modeling effort, pump characteristics and operations for the proposed Hughes Street pump station referenced the Downtown Drainage Improvements - Phase III Power Street Drainage Improvements New Pump Station and Interceptors design memorandum dated September 2007 with supplemental information received from Urban Engineering.

The Hughes Street pump station will be located at the intersection of Hughes Street and Mesquite Street. The pump station will convey contributing stormwater from a proposed dual 6' x 5' multiple box storm drain system along Hughes Street and Mesquite Street east and west of the pump station as shown in **Layout G2** in **Attachment G**. For modeling purposes, the operational pump station configuration (two sumps and seven pumps) and wet well volume were taken from the referenced design memorandum. The eighth submersible pump defined in the memorandum acts a backup to the other seven pumps during normal operation.

In addition, several drainage areas were modified to correlate with the proposed stormwater improvements, capturing runoff at proposed inlets along Mesquite Street from Harbor Drive to IH 37 as shown on **Layout G2**. Calculated composite CN's for the modified drainage area and corresponding XPSWMM runoff nodes are summarized in **Table C7** in **Attachment C**. Revised time of concentration calculations for modified drainage areas are summarized in **Table C8**.

A report showing the Flygt factory flow test results for the eight P7121 submersible pumps was provided by Urban Engineering. A single pump curve shown in **Table 21** was incorporated into the hydraulic model and assumed to be identical for all seven pumps. A Flygt factory flow test (

**Table** 22) for the NP3202 sump pump (provided by Urban) was referenced for the pump curve data associated with the two sump pumps operating to keep the wet well dry.

**Table 21 – Hughes Street Pump Curve**

| TDH (ft) | Single Pump Flow | |
|---|---|---|
| | (gpm) | (cfs) |
| 29 | 61,000 | 135.92 |
| 28 | 61,900 | 137.92 |
| 26 | 63,500 | 141.49 |
| 24 | 65,000 | 144.83 |
| 22 | 66,250 | 147.62 |
| 20 | 67,500 | 150.40 |
| 18 | 69,000 | 153.74 |
| 16 | 70,000 | 155.97 |
| 14 | 71,250 | 158.76 |
| 12 | 72,250 | 160.98 |

*TDH: Total Dynamic Head*

**Table 22 – Hughes Street Sump Curve**

| TDH (ft) | Single Pump Flow | | TDH (ft) | Single Pump Flow | |
|---|---|---|---|---|---|
| | (gpm) | (cfs) | | (gpm) | (cfs) |
| 81 | 0 | 0.00 | 30 | 2,900 | 6.46 |
| 80 | 40 | 0.09 | 25 | 3,200 | 7.13 |
| 75 | 250 | 0.56 | 20 | 3,450 | 7.69 |
| 70 | 450 | 1.00 | | | |
| 65 | 750 | 1.67 | | | |
| 60 | 1,100 | 2.45 | | | |
| 55 | 1,450 | 3.23 | | | |
| 50 | 1,800 | 4.01 | | | |
| 45 | 2,150 | 4.79 | | | |
| 40 | 2,400 | 5.35 | | | |
| 35 | 2,700 | 6.02 | | | |

*TDH: Total Dynamic Head*

Pump operations for the Hughes Street pump station were obtained through coordination with Urban Engineering. The pump operations were included to hydraulically simulate pump effectiveness within the hydraulic model. **Table 23** summarizes the pump operations for the Hughes Street pump station. As previous stated, limitations within XPSWMM pump controls do not allow starting elevations to be lower than stopping elevations. Similar to the Kinney Street pump station, the pump operation trigger elevations were modified so pumps would operate as intended. The modified pump operations, as shown in **Table 23**, were included to hydraulically simulate pump effectiveness within the hydraulic model.

Based on the normal operation of the sump pumps, the wet well was assumed to be dry initially (depth of 0 feet), which equated to the wet well invert of -18.25' NAVD 88. An initial depth of zero accounted for the activity of the two sump pumps.

**Table 23 – Hughes Street Pump Operations**

| Pump | Pump Status | Elevation NAVD 88 (ft) | Mod. Elevation NAVD 88 (ft) | Wet Well Depth (ft) |
|---|---|---|---|---|
| Lead Sump | On | -17.50 | -17.50 | 0.75 |
| | Off | -16.50 | -18.25 | 1.75 |
| Lag Sump | On | -16.50 | -16.50 | 1.75 |
| | Off | -18.25 | -16.51 | 0.00 |
| Pump 1 | On | -8.25 | -8.25 | 10.00 |
| | Off | -3.25 | -11.00 | 15.00 |
| Pump 2 | On | -7.25 | -7.25 | 11.00 |
| | Off | -4.25 | -8.25 | 14.00 |
| Pump 3 | On | -6.25 | -6.25 | 12.00 |
| | Off | -5.25 | -7.25 | 13.00 |
| Pump 4 | On | -5.25 | -5.25 | 13.00 |
| | Off | -6.25 | -6.25 | 12.00 |
| Pump 5 | On | -4.25 | -4.25 | 14.00 |
| | Off | -7.25 | -5.25 | 11.00 |
| Pump 6 | On | -3.25 | -3.25 | 15.00 |
| | Off | -8.25 | -4.25 | 10.00 |
| Pump 7 | On | -2.25 | -2.25 | 16.00 |
| | Off | -11.00 | -3.25 | 7.25 |

**Figure 13** graphically shows the original pump operations versus the modified operations.

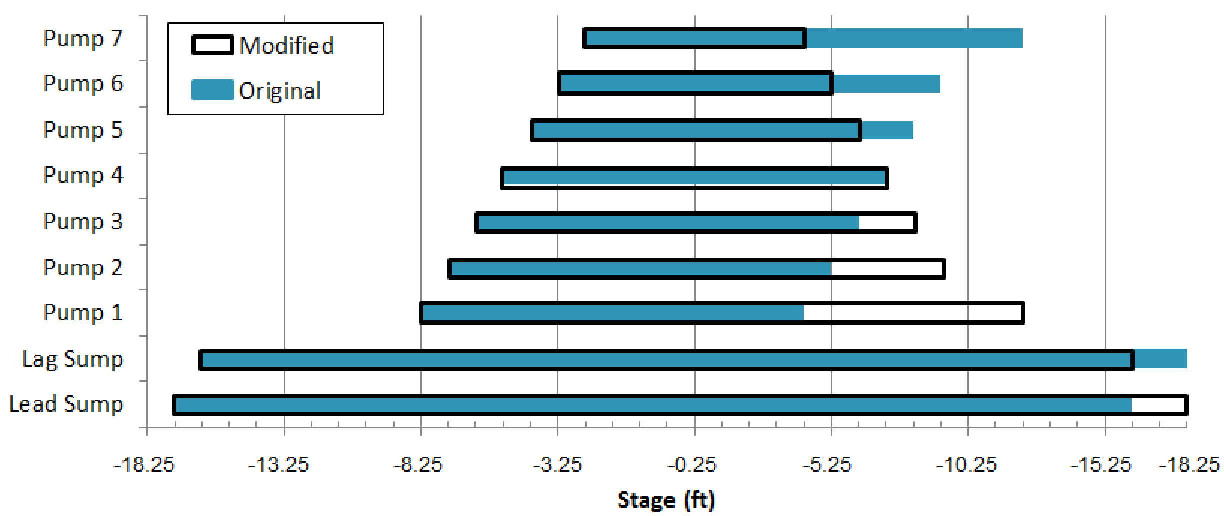


**Figure 13 – Hughes Street Pump Operations Schematic**

The proposed pump station and associated storm drain improvements were incorporated into the XPSWMM model to create the proposed Hughes Street pump conditions model. The model was executed for the 1% AC event at current sea level conditions to quantify the additional benefit produced by the project. After the initial run, it was determined Pumps 6 and 7 did not engage during the simulation as the WSEL within the pump station wet well did not exceed the specified stage in **Table 23**. Therefore, these two pumps were removed from the model for simplification.

Maximum WSELs were compared to existing conditions and summarized for each major Downtown drainage system in **Table C9** in **Attachment C** and approximate flood inundation limits of the entire study area are shown in **Exhibit B12** in **Attachment B**.

With the addition of the Hughes Street pump station, flood inundation depths were considerably reduced (1' to 2') within the northern Downtown/Port area, removing several buildings from being "at risk" including five surveyed buildings (No.15 to 19) listed in **Table 20**. Flooding west of Highway 181 within the Salt Flats area reduced from 0.5' to 1.0'; however, the Corpus Christi Wastewater Treatment Plant (Building 14) was shown to still be "at risk." Additionally, flooding was shown to be reduced by as much as approximately 0.5' along Water Street and Chaparral Street from Power Street to Taylor Street. This reduction can be attributed to local runoff along Mesquite Street being redirected to the proposed 9'x6' single box storm drain as shown on **Layout G2** instead of being conveyed to the system along Water Street.

## 9.3 Potential Improvement Option 1

As stated in Section 8, the twenty buildings selected for detailed survey coupled with the established existing conditions 1% ACE inundation mapping were used to define the design objective for the potential improvement options. Design constraints were defined as removing selected buildings from being considered "at risk" (inundation depths greater than 0.5'). Option 1 improvements were developed for two areas (northern Downtown/Port area and the Kinney Street-Water Street intersection) in order to improve flood protection for the surveyed locations/buildings listed in **Table 24**.

**Table 24 – Option 1 Design Constraint Building List**

| Bldg No. | Description | Bldg No. | Description |
|---|---|---|---|
| 2 | U&I Restaurant | 17 | Institution of Cultural Hispanica |
| 14 | CC Wastewater Treatment Plant | 18 | Texas State Museum of Asian Culture |
| 15 | Heritage Park - Galvan House | 19 | Harbor Playhouse |
| 16 | Heritage Park - Sidbury House | 20 | CC Hispanic Chamber of Commerce |

As shown in **Layout G3** in **Attachment G**, Buildings 14 to 20 were considered to be "at risk" in existing conditions for the Downtown/Port area. The overall objective of Option 1 was to improve flood protection at these building locations; however, an emphasis was set on protecting the Corpus Christi Wastewater Treatment Plant (Building 14) based on discussions with the City at the August 10, 2016 Review Workshop.

Near the Kinney Street pump station, the U&I Restaurant (Building 2) at the Kinney Street – Water Street intersection has been susceptible to frequent flooding. Improvements to drainage capacity in the area have been made with the construction of the Downtown Drainage Improvements Phase 1 – Project A which included a new 8' x 5' single box storm drain along Water Street and additional inlet capacity.

However, existing conditions inundation flooding is controlled by the current Kinney Street Pump Station tailwater conditions, and undersized storm drain laterals along the side streets.

Through evaluating the existing conditions results, it was determined that improvements within the Port Area could potentially improve the flood risk for Buildings 15 to 20, but would have relatively no benefit to the WWTP (Building 14) in the Salt Flats area. The same situation would occur for improving the drainage system within the Salt Flats area with regards to flood risk improvements between Building 14 and Buildings 15-20. Additionally, it was determined that Building 2 near Kinney Street was susceptible to flood risk due to local street flooding along Kinney Street and overall system surcharge along Water Street. Based on the existing conditions flooding assessment, an evaluation of potential large-scale and smaller-scale improvements was performed in order to inform the City of the breadth of system change considerations. Each system improvement scenario and assessment is discussed in more detail in the following sections.

***Evaluation of Possible Large-Scale System Improvements***

In evaluating the Port Area (Buildings 15 to 20), flooding was attributed to undersized storm drain systems and possible insufficient pump station capacity at the Power Street Pump Station. To improve the flood risk in this area, the following improvements were assessed:

- Upsize Water Street 7’ horseshoe pipe (from north) to 8’x7’ box.
- Upsize all Hughes Street lateral pipes contributing to new Water Street box to the maximum possible diameter.
- Add two additional pumps to the Power Street Pump Station.
- Modify timing of three existing Power Street pumps so they turn on at lower levels.

A review of the results found maximum hydraulic grade line (HGL) elevations in the Water Street system lowered by 2.3’ at the downstream end; however, Buildings 15 to 20 continue to be inundated by at least 0.5’. Based on the minimal flood reduction benefit associated with the significant enhancements to the existing pump station and trunk lines, the overall cost/benefit ratio would be extremely low. The Hughes Street Pump Station project scenario, evaluated in this study, provided an overall better flood reduction impact for Buildings 15 to 20.

For the Salt Flats area (Building 14), flooding to the WWTP was found to be due to the undersized drainage ditches inability to store local runoff until it could be released into the Salt Flats channel. All drainage systems in the Salt Flats area drain to discharge outlets at the Salt Flats Levee. However, the tailwater within the Salt Flats channel controls these outlet structures, preventing the local runoff from being discharged until the tailwater surge has subsided. In order to alleviate flooding in the area, i the following improvements were incorporated within the Salt Flats area and Downtown:

- Construct lift station along Salt Flats levee (near Brewster Street) with three low head pumps (equivalent to Fairbanks Morse Pump 30” 8211 series) to operate after the levee gate is closed.
- Channel widening upstream of the lift station, including the ditch just north of the City WWTP along Resaca Street. The existing ditches (approx. 10’ bottom width, 6:1 side slopes) between the levee and Sam Rankin Street are excavated to a 25’ bottom width with 3:1 side slopes, keeping the existing depth. The ditch adjacent to Resaca Street is excavated to 18’ bottom width, 2’ deep with 3:1 side slopes.
- Install relief 2-6’x3’ box lateral along Resaca Street to convey runoff from the City WWTP ditch (near Tancahua Street / Resaca Street) east to the existing Water Street horseshoe storm drain trunk line and out through the Power Street Pump Station.

- Add two additional pumps to the Power Street Pump Station.
- Modify timing of three existing Power Street pumps so they turn on at lower levels.

Results showed that with these improvements in place the 1% ACE flood extents no longer inundated the WWTP, and minimal flood depths are computed in the Port Area near Buildings 15 to 20. The lift station at the Salt Flats Levee and channel widening improvements alone did not provide sufficient capacity to eliminate flood risk for the WWTP. Although the WWTP is removed from the 1% ACE inundation extents when all improvements are implemented, construction costs would likely significantly exceed the benefit of providing flood relief for one structure location (Building 14). With a potentially low benefit/cost ratio, this improvement option would not be recommended.

In the Downtown area near the Kinney Street – Water Street intersection, the following improvements were identified to eliminate flood risk for Building 2 as well as reduce flooding within the Water Street "low" at the Kinney Street Pump Station:

- Modify timing of four existing Kinney Street pumps so they turn on at lower levels.
- Add one additional pump to the Kinney Street pump station.
- Upsize lateral on Kinney Street adjacent to Building 2 from 24" to 36" pipe.
- Upsize lateral on Water Street from 36" pipe to 5'x3' SBC.
- Add a second 8'x6' box to the Kinney Street Pump Station discharge line.

The 1% ACE flood inundation extents were removed from Kinney Street (west of Water Street) and adjacent to Building 2, but remain along Water Street (depths greater than 0.5'). Flooding along Kinney Street appeared to eliminate the local side street flooding, improving flood risk for Building 2, without significant downstream impacts to the system within Water Street. Flooding within Water Street was minimally impacted by the significant amount of improvements to the storm drain system and pump station. It appears that extensive improvements will only have negligible benefits along Water Street and are unlikely to eliminate flooding.

### *Evaluation of Possible Small-Scale System Improvements*

The smaller-scale system changes discussed in this section for the Salt Flats and Kinney Street areas present more feasible alternatives and were incorporated into the existing conditions 1% ACE 1D/2D XPSWMM model. See **Layout G3** for a schematic of proposed recommended improvements at each specified area.

From assessments of the large-scale improvements, it was determined that improvements to the Downtown storm drain system and Power Street Pump Station had insignificant benefits for the WWTP. In considering construction feasibility, the following improvements were simulated to improve the flood risk of Building 14.

- Construct lift station along Salt Flats levee (near Brewster Street) with 3 low head pumps to operate after the levee gate is closed.
- Channel widening upstream of the lift station, including the ditch just north of the City WWTP along Resaca Street.
- Install relief 2-6'x3' box lateral along Resaca Street to convey runoff from the City WWTP ditch (near Tancahua Street / Resaca Street) over to the existing Water Street Horseshoe and out through the Power Street Pump Station.

As previously stated, the Building 2 flood risk was attributed to local street flooding on Kinney Street west of Water Street. Local storm drain lateral improvements listed below were simulated without modifying the Kinney Street Pump Station:

- Upsize lateral on Kinney Street from 24” to 36” pipe.

The proposed Option 1 scenario model was simulated. Maximum WSELs for the Recommended Small-Scale Improvements were compared to existing conditions and summarized for each major drainage system in **Table C10** in **Attachment C**. Approximate flood inundation limits of the entire study area are shown in **Exhibit B13** in **Attachment B**.

As shown in **Exhibit B13**, only two of the buildings defined in the design constraints had improved flood protection, with one (No. 2) being completely removed from being “at risk” under modeling conditions for the 1% ACE. Flooding remained within Water Street, although depths greater than 0.5’ were confined to the streets. Based on the design constraint, the storm drain lateral improvement is likely to provide the most cost effective benefit to the Kinney Street area; more extensive improvements are unlikely to provide significant additional benefit.

The flood protection at the WWTP (Building 14) was improved substantially through the installation of a lift station and dual relief 6’x3’ storm drain box. The 1% ACE flood extents were considerably reduced within the Salt Flats area with flood depth reductions from 1.6’ to 0.6’ along Resaca Street near the WWTP. Additionally, flood depths near the proposed lift station at the Salt Flats Levee were reduced by an average of 0.8’. However, there continue to be several areas of flooding greater than 0.5’ adjacent to the WWTP which can be attributed to drainage capacity limitations within the Salt Flats/Port areas.

## 9.4 Potential Improvement Option 2

With a design approach similar to Option 1, Option 2 improvements were developed with the design objective of improving flood protection for the Education Service Center Region 2 (Building 5) near Chaparral Street and John Sartain Street. As shown in **Exhibit B14**, Building 5 is considered “at risk” due to flooding along Chaparral Street and William Street. A review of existing conditions showed local flooding to be caused by surcharging of the storm drain system along Lawrence Street and resulting conveyance of storm water southward along Chaparral Street. The 24” to 30” RCP storm drain system bypasses the existing 60” RCP within Water Street and discharges into Corpus Christi Bay at the Seawall.

In order to satisfy the design constraint, the following improvements were incorporated into the existing conditions 1% ACE 1D/2D XPSWMM model (see **Layout G4** for schematic of proposed improvements):

- Upsize existing 24” to 27” RCP from Chaparral Street to Water Street to 48” RCP.
- Tie upsized lateral system to existing 60” RCP within Water Street.

Maximum WSELs were compared to existing conditions and summarized for each major drainage system in **Table C11** in **Attachment C** and approximate flood inundation limits of the entire study area are shown in **Exhibit B14** in **Attachment B**.

Surcharging from the proposed storm drain system along Lawrence Street was considerably reduced in the model, reducing surface inundation depths by a maximum of 0.7’. As a result, flooding along Chaparral Street was eliminated, removing Building 5 from being “at risk” under modeling conditions for the 1% ACE. Increasing storm drain capacity to the Lawrence Street system had negligible impacts (less than 0.1’) on the existing 60” RCP within Water Street. The proposed improvements were shown to have localized benefits only, focusing on the insufficient storm drain capacity along Lawrence Street.

# 10 Conclusion and Recommendations

A holistic approach to evaluating the Downtown Flood Protection system was developed to better understand the capacity of the City's existing drainage system as well as develop a more accurate representation of potential flood risk within the FEMA defined "Seclusion Zone." For this study, a hydrologic and hydraulic 1D/2D XPSWMM numerical model was developed, incorporating the major existing drainage systems including the Power Street and Kinney Street pump stations. In addition, a wave overtopping analysis was completed for the Seawall and considered in the evaluation. The developed existing conditions model provides a comprehensive assessment of the Inner Harbor watershed, further advancing the level of detail from previous watershed studies. In addition, this study provides several benefits to the City, including detailed floodplain mapping within the Seclusion Zone, characterization of the existing level of flood protection for the Downtown Bayfront area, and a future level of protection assessment which includes the potential effects of sea level rise as discussed herein.

***City Benefits***

In developing the watershed-wide model, a baseline condition of the Downtown Flood Protection system is established which benefits the City in several ways:

- The model helps define the level of flood protection currently provided for the Downtown Bayfront area (see **Figure 6** through **Figure 9**).
- If the model is accepted by FEMA, the City will maintain local ownership of the effective model used to produce effective floodplain in the Downtown Bayfront area.
- The effective model would be readily available for the City to provide to developers/consultants.
- The study provides a comprehensive inventory of major drainage systems within the watershed.
- The model provides the ability for the City to identify existing system issues, and can be applied for planning-level development concepts and master plans.
- The model will allow the City to thoroughly evaluate proposed flood protection improvements, including improvements along the Seawall, upgrades to the pump stations, and additional storm drain improvements in the protected area.

***Comparison to FEMA Results***

As previously stated, the XPSWMM model developed for the current study included the same 2011 LiDAR terrain data as FEMA's Nueces County Pilot LAMP FLO-2D model. However, several important model improvements were incorporated as noted below:

- Hydrology updated using the local City of Corpus Christi Drainage Design Manual (DDM).
- Drainage area delineation level of detail refined to the local storm drain/channel level.
- Includes the major storm drain systems (greater than 24") and channel systems.
- Runoff conveyed through local drainage systems to the storm water pump stations, accounting for subsurface system storage and routing or attenuation effects.
- Pump station data refined to include all pumps with original pump curves and current procedures and operational elevations.
- Wave overtopping analysis completed to update estimates of additional flow contribution from waves overtopping the Seawall.

As a result, the developed floodplain limits within the downtown "Seclusion Zone" greatly differ from the floodplain established in FEMA's LAMP study, except along Shoreline Boulevard directly adjacent to the Seawall. Referencing Error! Reference source not found. **11** and **Figure 12,** shallow flooding along Shoreline Boulevard caused by wave overtopping of the Seawall was defined as a Zone AO with average depths of 1 foot. However, the XPSWMM model showed considerably less flooding within the Downtown area. As shown in **Figure 11**, the XPSWMM model suggests the majority of the Seclusion Zone south of Power Street should consist primarily of Zone AO with average depths of 1 foot instead of the static BFE of 8 feet and 9 feet NAVD 88 shown by FEMA in **Figure 12**.

As developed in the FEMA LAMP study using the “natural valley” approach, which assumes levee breaching, all areas within the Seclusion Zone below El. 10 feet NAVD 88 and not designated as Zone AE or Zone AO were proposed to be mapped by FEMA as Zone D (**Figure 12**). This zone designation was appropriate for the LAMP study based on the level of detail included in the FLO-2D model. However, with the development of the current XPSWMM model, the area has been studied in more detail providing an improved representation of potential flood risk within the Seclusion Zone. Since these areas of flood risk are no longer undetermined based on the current study, the zone designation is proposed to be mapped as a Shaded Zone X based on the improved confidence in the potential flood risk.

If FEMA insists on using the Zone D designation, the zone extents should be mapped with the XPSWMM model developed in this study using the “natural valley” approach, with the pump stations in operation. With a more detailed representation of the Seclusion Zone drainage infrastructure, expected WSELs would be lower than the El. 10 feet NAVD 88 currently identified in the LAMP study.

***Future Sea Level Rise***

Referencing **Table 13**, wave overtopping flow rates significantly increased for the 1% ACE for the 100-year RSLR condition. With overtopping flow rates twenty to sixty times greater than at current (2016) sea level conditions, maximum WSELs for the 1% ACE increased 0.5 foot along Shoreline Boulevard and up to 5 feet within the Downtown area. Including the effects of RSLR would result in floodplain limits similar to the FEMA FLO-2D extents shown in **Figure 12**. However, WSELs would be at a consistent static BFE of 8 feet NAVD 88. Shallow flood depths along Shoreline Boulevard and side streets west to Water Street would range from 1 foot to 3 feet.

***Drainage System Improvements***

From the Pump System Alternatives Analysis, it was determined that implementing modifications to the existing pump stations alone is unlikely to create noteworthy reductions in flooding. Large scale improvements to interior drainage systems including pump station modifications would be necessary to provide a measurable decrease in flood depths and improve overall flood protection. However, areas of isolated concern with one to two buildings considered “at risk” could potentially be mitigated by improving localized storm drain system capacity, most notability lateral storm drain systems along the east-west corridors.