Texas Coastal Protection and Restoration Project - DRAFT Interior Drainage Hydrology and Hydraulic Report

August 31, 2018

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Executive summary

The Tentatively Selected Plan (TSP) for the Coastal Texas Protection and Restoration Study calls for erecting a coastal flood barrier along portions of Galveston Island and Bolivar Peninsula, as well as a number of flood risk reduction features interior to the bay. The proposed plan will reduce risk to inundation from tidal surge, but also restricts overland and fluvial flows from freely entering the bay. In support of the Tentatively Selected Plan, a hydrologic and hydraulic evaluation was conducted to estimate the required facilities to mitigate the impacts of proposed risk reduction features (coastal barriers) on the fluvial and overland flows at Clear Creek, Dickinson Bayou, and the City of Galveston

The analysis was based on coupled hydrologic and hydraulic models of the three (3) drainage areas of interest. For Clear Creek and Dickinson Bayou, the hydrologic modeling was conducted in Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) and the hydraulic modeling and pump sizing was conducted in unsteady Hydrologic Engineering Center River Analysis System (HEC-RAS). For the City of Galveston, both the hydrologic and hydraulic modeling was conducted in the Environmental Protection Agency Storm Water Management Model (EPA SWMM). The analysis included the evaluation of five storm return periods: 10, 25, 50, 100, and 500-year precipitation events. Based on discussions and guidance from the United States Army Corps of Engineers (USACE), the precipitation depths and distributions were taken from the Harris County Flood Control District (HCFCD) Hydrology and Hydraulic Manual, with the total rainfall depths augmented by 30% to reflect the anticipated revisions to the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 Volume 9 data. Within each watershed, use was made of existing storage by dewatering the interior area in advance of the storm and by allowing the interior water surface to rise to a predetermined elevation to attenuate peak flow without causing damages.

The unsteady state HEC-RAS models provide more refined estimates of the required pumping rates and additional facilities to mitigate the impacts of the coastal barrier during precipitation events. Since there are no fluvial watercourses in Galveston and since consolidation piping would be required, SWMM was appropriate to estimate the required facilities. The hydrology portion of the Galveston SWMM model was tested against other theoretical run-off calculation methods to do a rudimentary level of validation. The Galveston SWMM model was also further modified from the TSP layout by adding in two additional pump site locations and conveyance channels to reduce the required pump and pipe sizes calculated during the preliminary analysis.

The hydrology and hydraulic models were used to develop the design facilities for the 25-year (+30%) fluvial event in combination with the overtopping rate associated with the 100-year tropical storm. A summary of the resulting design facilities is provided for each site in the following tables. Note that due to uncertainties regarding the final design of the proposed seawall improvement, the pump facilities for Galveston assume no overtopping along the flood barrier. Once a final seawall configuration is agreed upon, overtopping rates should be included in the pump designs.

Table 1. Final estimated pump size for Clear Creek.

Rainfall Return Period	Pumping Rate (cfs)	Flood Wall Height (ft NAVD88)	Gate width (ft)	Sill Elevation (ft NAVD88)
25-yr+30%	45,661	17	75	-12

Table 2. Final estimated pump size for Dickinson Bayou.

Rainfall Return Period	Pumping Rate (cfs)	Flood Wall Height (ft NAVD88)	Gate width (ft)	Sill Elevation (ft NAVD88)
25-yr+30%	19,125	18	100	-9

Table 3. Final estimated facilities for the 25-yr+30% rainfall event for Galveston.

Rainfall Return Period	Location	Conveyance Channel Dimensions (ft)	Pumping Rate (cfs)	Max Flood Water Elv. In Bayou (ft NAVD88)
25-yr+30%	Offatts Bayou (Pump Site No. 1)	30’ wide x 13’ high	250	2.5*
25-yr+30%	Pump Site No. 2	20’ wide x 10’ high	1,500	-
25-yr+30%	Pump Site No. 3	20’ wide x 10’ high	4,500	-
25-yr+30%	Pump Site No. 4	20’ wide x 10’ high	1,500	-

*Assumes Offatts Bayou water level is pumped down to -1 ft NAVD88 (which is approximately 1 ft below mean low water) prior to the storm.

Additional operational scenarios were also tested to complete the range of storms prescribed by the USACE for the project (10-, 25-, 50-, 100-, and 500-year storms, as well as evaluating the system’s performance during a rainfall series derived from Hurricane Harvey ***Note Harvey Results will be presented in the Final Draft***). Resulting inundation levels with the designed pumps in place for the 50-, 100-, and 500-year rainfall events were also investigated.

This report and associated evaluation are an interim step in advancing the planning for the Coastal Protection Project. Accordingly, within this report, recommendations are provided for advancing the analysis beyond the scope of this project phase to enable the advancement of the future project. A summarized list of the recommendations is provided as follows:

- Calibrate the Clear Creek hydrologic and hydraulics model to Hurricane Harvey
- Conduct a detailed field campaign at Dickinson Bayou for refining and confirming flow patterns and sub-watersheds boundaries is recommended for the next level of effort for hydrology analysis.
- Install gauges within Dickinson Bayou to collect metered flowrate data for calibrating and validating the Dickinson HEC-HMS model;
- Complete a field camping in Dickinson Bayou, including cross-section topographic data and Manning’s n, to update the HEC-RAS model with data representative of 2018;
- Research into historical flood marks along Dickinson Bayou is recommended to calibrate the hydraulic model.
- Install gauges within the Galveston internal drainage system to collect metered run-off data for calibrating and validating the Galveston SWMM model;
- After the Galveston seawall design is finalized, evaluate the impact of surge/wave overtopping on the design pump facilities;

1 Purpose

The Tentatively Selected Plan for the Coastal Texas Protection and Restoration Study calls for erecting a coastal flood barrier along portions of Galveston Island and Bolivar Peninsula. This barrier will reduce risk to inundation from storm surge. Storm surge is driven by winds, atmospheric pressure differences and astronomical tides. Two of these factors, wind and low atmospheric pressures accompany tropical and extra-tropical storm events which also produce extreme rain events. Without proper measures, coastal flood defenses intended to mitigate flooding resulting from storm surge can form a barrier to runoff generated during the event. The resulting flooding on the protected side of the flood barrier can create substantial flood damages that could potentially exceed the damages cause by the surge. This is particularly true when the flood barrier traverses a fluvial watercourse. In this case, even when the navigation gates are in the open position, the hydraulic constriction they create could induce flooding for rainfall events that are not accompanied by surge. There are three measures typically implemented to mitigate these effects:

1. Create or utilized existing storage on the protected side of the coastal barrier to store excess flow for release following the storm, or to attenuate peak flows to enhance the performance of other mitigation measures.
2. Employ sluice (lift) gates to augment the navigation opening during precipitation events not accompanied by surge, and potentially maximize gravity discharge during surge events.
3. Employ pumps to convey runoff over the barrier, pumps may be employed during precipitation events as well as precipitation events accompanied by surge.

The intent of this report is to establish preliminary (order-of-magnitude) values for the size of facilities necessary to mitigate the effects of the coastal barrier on interior drainage. The three locations to be evaluated as part of this study are:

- Clear Creek
- Dickinson Bayou
- City of Galveston
  - Offatts Bayou
  - North Galveston Pumping Stations (areas not draining to Offatts Bayou)

For Clear Creek, Dickinson Bayou and Offatts Bayou the evaluation will include scenarios when the surge and precipitation are coincident and the effects of a rainfall only event that occurs with the navigation gates open. For the northern portion of Galveston Island, there is no navigation channel and thus no navigation gate. Ultimately, there will be gravity outlets, which will likely be modifications of the existing stormwater outfalls equipped with tide gates and sluice gates. However, sufficient information on the existing drainage system was not available to evaluate these facilities during this analysis. The evaluation and operational scenarios are discussed in more detail in their respective sections.

2 Data Overview

This section addresses the general data sources and references that were collected in support of the interior drainage analysis. Site specific data is addressed within the respective sections.

2.1 GIS Data

GIS data were collected from a variety of sources described in following sub-Sections.

2.1.1 Soils Data

Soil GIS data for Harris, Galveston, Chambers and Brazoria counties was collected from the SSURGO Database (NRCS, 2014), which contains information about soil collected by the National Cooperative Soil Survey (NCRS). These data were collected from the United States Department of Agriculture (USDA) Web Soil Survey portal (NRCS, 2009). Furthermore, two more general soils datasets: “Major Land Resource Areas in Texas” and “Coordinated Common Resource Areas in Texas” were also acquired from NCRS.

2.1.2 Land Use Data

Land use and land cover data can be used for developing or updating watershed models. Original Texas Land Survey (OTLS) data was acquired from the Texas General Land Office (TxGLO) GIS web portal for the state of Texas. These surveys were compiled by the GLO and shows the original land grants and bay area tracts. Land cover data for the state of Texas was also acquired from the USGS National Land Cover Database (NLCD) based on classified 2011 Landsat satellite imagery data. Finally, the land use data from the Houston-Galveston Area Council (HGAC) was acquired from the Regional Land Use Information System (RLUIS). The land use data show the spatial distribution of land use into 11 different categories (Commercial, Industrial, Residential, Government/Medical/Education, Multiple, Other, Parks/Open Spaces, Vacant Developable, Undevelopable, Unknown, and Undetermined).

2.1.3 Topographic Data

Topographic information was obtained for the Galveston Island site through the use of NED files. The data file “USGS NED 1/3 arc-second n30w095 1x1 degree ArcGrid 2018” (USGS, 2018) was downloaded from the USGS National Map website (https://viewer.nationalmap.gov/launch/). This data was trimmed to the Galveston site area and converted to 1 ft contours.

LiDAR data for all three project sites were acquired from the Texas Natural Resources Information System (TNRIS) data download portal in ¼ quadrangle georectified 1-meter resolution DEM images (TNRIS, 2018). The LiDAR surveys for the Clear Creek watershed and part of the Dickinson Bayou watershed were performed by the HGAC in 2008, and the surveys for the remaining portion of Dickinson Bayou and Galveston watersheds were performed by the Federal Emergency Management Agency (FEMA) in 2006.

2.2 Tidal Data

Tidal elevations, for Dickinson Bayou and Clear Creek, were obtained from NOAA gage 8771013 at Eagle Point, TX; the NAVD88 datum conversation was obtained from NGS (NGS, 2017).

Tidal data for the Galveston site was collected from the NOAA Tidal Station No. 8771486, Galveston Railroad Bridge, TX (NOAA, 2018). This Tidal Station is located in West Bay, approximately two miles north-northwest of the Offatts Bayou mouth.

2.3 Reference Documents

The following documents provided guidance or general reference information:

Report on the Interior Hydrology and Hydraulic Analysis for the GIWW West Closure Complex., Last Updated 29 October 2014; Prepared by: John Boeckmann, CEMVS-EC-GW

Flood Insurance Study, Galveston County, Texas, Revised Preliminary February 28, 2018; FEMA

Flood Insurance Study, Harris County, Texas; Revised January 6, 2017; Federal Emergency Management Agency

Hydrology and Hydraulics, Guidance Manual; December 2009; Harris County Flood Control District

Urban Hydrology for Small Watersheds; Technical Release 55, June 1986; United States Department of Agriculture

Hydrological Analysis of Interior Areas, (EM 1110-2-1423); 15 January 1987; U.S. Army Corps of Engineers

Additional reference documents are provided in Section 9.

3 Rainfall Analysis

A rainfall analysis, using existing data, was performed to determine the appropriate precipitation conditions for the interior drainage analysis for Clear Creek, Dickinson Bayou, and the City of Galveston. This section summarizes the analysis findings and establishes the precipitation magnitude and distribution that will be applied to the interior drainage analysis for the Texas Coastal Protection and Restoration Project.

3.1 Data Collection

Existing extreme rainfall data was collected from various sources, including the following:

1. Hershfield, D.M., 1961. Rainfall Frequency Atlas of the United States, Technical Paper (TP) No. 40. Weather Bureau, U.S. Department of Commerce, Washington, D.C.
2. USGS, 2004. Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas. Report No. FHWA/TX-04/5-1301-01. U.S. Department of the Interior, U.S. Geological Survey
3. Harris County Flood Control District (HCFCD), 2009. Hydrology & Hydraulics Guidance Manual.
4. TX Tech University. 2015. New Rainfall Coefficients -- Including tools for estimation of intensity and hyetographs in Texas. Report No. FHWA/TX-15/0-6824-1. Texas Tech University, Water Resources Center.

In addition to the above sources, Mott MacDonald also sought access to the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 preliminary rainfall values for Texas in March 2018; however, due to the preliminary nature of this data, NOAA was unable to share this information with Mott MacDonald.

Given the lack of recent data, Mott MacDonald requested USACE assistance in obtaining the preliminary data from NOAA. From the USACE's experience as a member of the NOAA Atlas 14 Volume 9, update review team, the USACE advised Mott MacDonald to increase the extreme rainfall values from Technical Paper 40 (TP-40) by 30%, to approximate the rainfall values of the NOAA Atlas 14 preliminary results for Texas. The results of this analysis were transmitted to the USACE in the "Extreme Rainfall Data for Drainage Analysis Technical Memo" addressed to Dr. Himangshu Das, USACE, from Mott MacDonald, dated June 22, 2018.

3.2 Data Analysis

The extreme rainfall values from the sources listed in Section 3.1 were compared for the 10, 25, 50, 100, and 500-yr return periods for the study area. Initially, the 1,000-year event was requested, however, when the preliminary magnitude of the required facilities was presented to the USACE, the 1,000-year event was removed from the project scope by the USACE. The date of publication for each dataset was also taken into consideration. After evaluating the existing data, it was determined that the rainfall values provided by the Harris County Flood Control District (HCFCD) Hydrology & Hydraulics Guidance Manual would be used for the interior drainage analysis at Clear Creek, Dickenson Bayou, and the City of Galveston. This was decided because a comparison of the Harris County 24-hour rainfall totals for various return periods in the HCFCD Hydrology & Hydraulics Manual showed the data was consistent with the TP-40 (1961) data (see Table 4). This allows the current work to be consistent with other established evaluations within the project area.

Table 4. Comparison table of the 24-hr rainfall depths from HCFCD (2009) and TP-40. Dashes indicate no data is provided by the source for that return period.

Return Period (years)	HCFCD (2009) 24-hr rainfall depth (inches)	TP-40 24-hr Rainfall Depth (inches)
10	7.8	8
25	9.8	10
50	11.6	11
100	13.5	13
500	19.3	-

Since the NOAA Atlas 14 preliminary rainfall would have offered the most up-to-date data for the study, as instructed by the USACE, a 30% increase was applied to the rainfall values from the HCFCD Hydrology & Hydraulics Guidance Manual. Subsequently, it was requested that a 15% increase also be investigated. Table 5 provides the list of extreme rainfall values that are being used in the interior drainage analysis for this study.

Table 5. Extreme rainfall values from HCFCD (2009), with 15% and 30% increases for use in the interior drainage analysis.

Return Period (year)	Rainfall Depth (inches)		
	24-hr	24-hr + 15%	24-hr + 30%
10	7.8	9.0	10.1
25	9.8	11.3	12.7
50	11.6	13.3	15.1
100	13.5	15.5	17.6
500	19.3	22.2	25.1

Furthermore, the HCFCD 100-yr 24-hr rainfall distribution was compared to the Natural Resources Conservation Service (NRCS) 24-hr Type III distribution. This comparison is shown in Figure 1. The difference in peak intensity points stems from the criteria established in HCFCD (2009), which sets the intensity position at 67% (i.e. the peak intensity occurs 16 hours into the storm for a 24-hour storm.) This also results in a more gradual ramp up and shorter ramp down in the 100-year distribution. However, the general shape of the curve is similar to that of the NRCS Type III. Testing was performed using the 100-yr rainfall to compare the results from the distribution with the intensity position at 67% to the results from the distribution with the intensity position at 50% (which is more similar to the NRCS Type III curve), and the difference in results was found to be nominal. Thus, the analysis will continue using the HCFCD 100-yr 24-hr rainfall distribution curve since it is specified by HCFCD (2009) for the project site area.

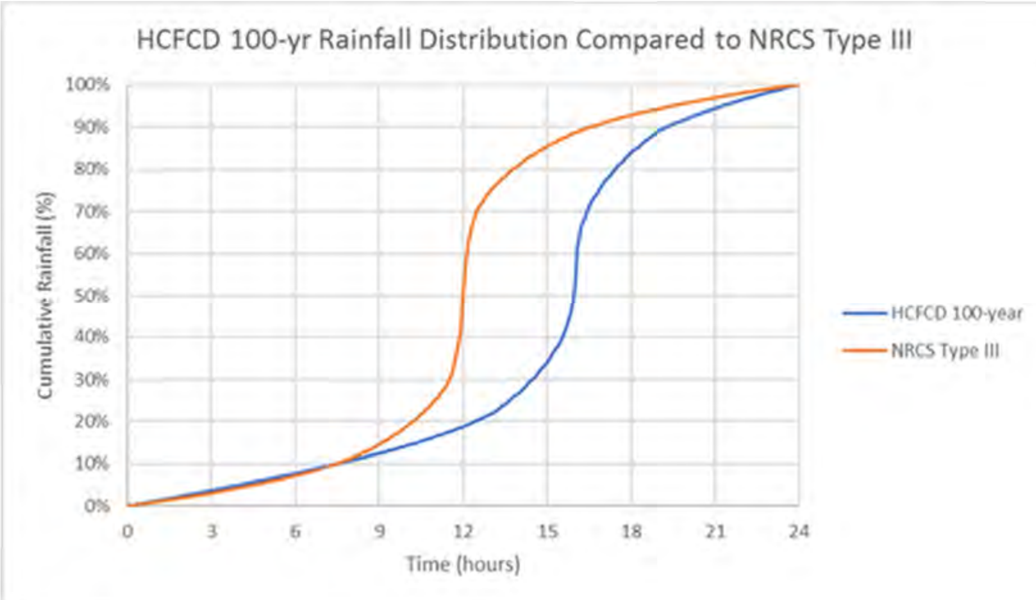

Figure 1. Comparison of rainfall distribution between the HCFCD 100-yr 24-hr rainfall and the NRCS Type III.

3.3 Rainfall Surge Coincidence

The primary purpose of the project is to prevent inland flooding from surge events during hurricanes, however most surge events are coincident with rainfall events. Measured tide data at NOAA-8771013 (Eagle Point) and measured daily rainfall data at USC00414333 (Houston National Weather Service Office), the peak daily water level and daily rainfall was plotted and is shown in Figure 2. Based on this figure, the peak historical surge event (Hurricane Ike) experienced a greater than 50-year surge, however it coincided with less than a 10-year rainfall. Similarly, the peak historical rainfall event (Hurricane Harvey) experienced a greater than 50-year rainfall, however it coincided with less than a 10-year surge. As can be seen by looking at Figure 2 the upper righthand corner of the graph is empty, demonstrating it is reasonable to conduct the evaluation assuming there is a relationship between surge and precipitation events, but not coincidence. Thus, it was determined that while extreme precipitation may occur during extreme surge events it is unlikely that a 100-year precipitation event would be coincident with at 100-year surge event.

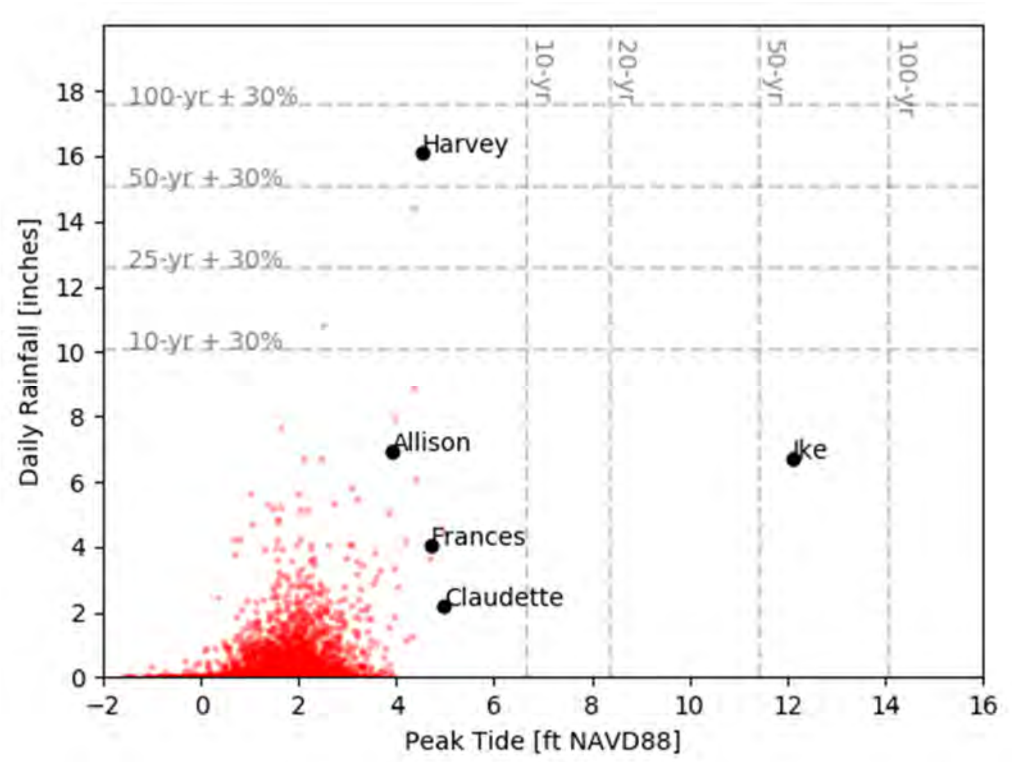

Figure 2. Peak daily water level at NOAA-8771013 vs. Daily Rainfall at USC00414333.

Through correspondence with USACE, it was agreed that the pump capacity would be designed for the 25-year rainfall condition, assuming that this rainfall would conservatively correspond to the 100-year surge during which the navigation gates would be closed, and the pumps would be solely responsible for draining the watershed. This design condition is similar to that adopted for the West Closure Complex in New Orleans, LA, which based the pumping rate on a 10-year precipitation return period. It is noted that during the West Closure Complex design process, the final decision on the level of protection was not finalized until later project phases.

3.4 Project Application

The interior drainage models for Clear Creek, Dickenson Bayou, and the City of Galveston have been tested with all three 24-hr rainfall magnitudes from Table 5 (24-hr, 24-hr + 15%, and 24-hr + 30%) to determine resulting required pump sizes. However, in coordination with USACE, final reporting and pump size estimation will be based on results from only the 24-hr + 30% rainfall values. Thus, this report only provides detailed modeling results from the 24-hr + 30% rainfall values.

4 Evaluation Procedure

The proposed facilities will be in place during all conditions that happen to occur, with the goal of providing storm surge damage reduction while avoiding adverse impacts during fluvial flooding events. Therefore, it is necessary to evaluate the facilities over a broad range of conditions in addition to the design flood protection condition. For example, while the facilities may be adequate for the 25-year precipitation coincident with the 100-year surge they may induce flooding for a 100-year precipitation only event. It may also impact daily tidal inundation. To be thorough, in this analysis, a series of evaluations were planned and conducted to cover the likely range of conditions that may occur. Obviously, there are unlimited combinations of events that may occur, however through consideration of reasonable events and the use of sensitivity analysis the likely combination of events can be addressed.

It was determined that additional facilities, beyond the design condition pumping facilities, were required to offset the impact of the surge barrier under the anticipated range of conditions. There will be subsequent phases to this. Since the rainfall associated with various return periods and the design return period may subject to revision during later project phases, a graph of pumping rate versus 24-hour rainfall depth is provided for each site.

4.1 Preliminary Analysis

A preliminary analysis was prepared and presented to the USACE in the Preliminary Report on July 18, 2018. The intent preliminary analysis was to appraise the USACE as to the magnitude of facilities required to meet the project's objectives. The secondary purpose of the preliminary analysis was to initiate the siting and cost estimation for the facilities. The third was to provide the general analysis methodology and procedure to subsequently by implemented in this report for review and comment. The procedures and results of the preliminary analysis are contained in the prior report, they are superseded by this report and the content of the Preliminary Report is not repeated herein

4.2 General Consideration

Prior to laying out the evaluation procedure, the treatment of certain items must be established, in this case coincidence of precipitation and surge, and seal level rise.

4.2.1 Precipitation and Surge Time Series Coincidence

The quantification of extreme water surface elevations is a central component in designing a storm surge gate. The storm surge results presented in this report are based on a Level 3 analysis which employs a probabilistic framework to encompass the range of possible variations in the storm conditions; such method is called the Joint Probability Method with Optimal Sampling (JPM-OS). The Joint Probability Method (JPM) approach describes a storm in terms that have the greatest influence on storm surge: (1) storm landfall location, (2) central pressure difference, (3) storm radius, (4) forward velocity, and (5) storm heading. Based on the joint probability distributions developed for the storm parameters, occurrence probabilities are assigned to each storm in the synthetic storm suite. The selected storms are then simulated using a coupled wave and surge numerical model.

The JPM-OS storm surge analysis conducted by the USACE as part of this study was used to complete the analysis presented in this report. A total of 20 synthetic storms were used by the USACE to calculate extreme value analysis on storm surge at the project site. The extreme surge values for the 2017 10-, 20-, 50-, 100-, 200-, 500-yr return period and the synthetic storm time series at Dickinson Bayou outlet are shown on Figure 3. It assumes the results from Dickinson Bayou are representative of Clear Creek and City Galveston since the storm surge study encompassed Galveston Bay in its totality.

The operation of the proposed structure, including opening and closing of the gates and activation of the pumps, depends on the time when the storm surge reaches the project site. Based on rage of potential storms at the project site, the time for the water surface elevation to recede from its peak back to MHW can be as short as 28 hr (1.2 days), or as long as 100 hr (4.2 days) as seen on Storm 161 and Storm 270, respectively. Due to the wide range of time for the storm surge elevation to recede to MHW, which was set as the boundary condition on the hydraulic model, it was concluded that in order to prevent damages from tail water effects associated with storm surge, it was conservative for the storm surge gates to remain closed during the entire time period of the fluvial event.

While active control of the gates could in theory be applied to maximize gravity flow it would increase project risk to rely on such an operational procedure to establish the project pumping rates. As shown above, it is probable that the gates may need to remain closed during the entire duration of a fluvial event. In additional, operating the gates based on the varying surge conditions during the storm increases the risk that a mechanical or operational failure may occur at a critical time, resulting in unnecessary damages.

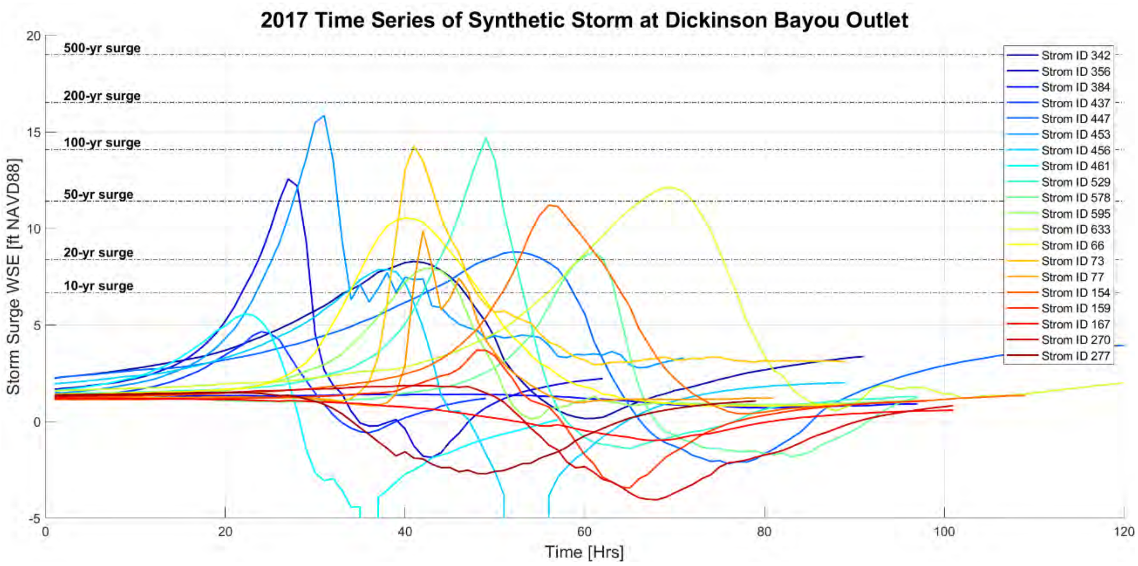

Figure 3. Storm surge elevation at the mouth of Dickinson Bayou based on 20 JPM-OS synthetic storms for 2017.

4.2.2 Sea Level Rise

Sea level projections were provided by USACE for the purpose of calculating forces on the navigation gates and associated structures. The interior drainage analysis likewise sought to account for sea level rise to evaluate the performance of the proposed facilities over the anticipated lifespan of the project. Sea level rise has the potential to impact the proposed facilities in three instances:

- The initial conditions with the watercourse – This would affect the amount of pumping required to dewater the watercourse in advance of the storm. Given the magnitude of pumping rates the additional water volume did not significantly alter the dewatering time, accordingly sea level rise was not explicitly evaluated when considering pre-storm dewatering.

- Targeted interior water surface elevation – As discussed later MHW was used as a target elevation to limit interior water surfaces during the design condition and the fluvial water level was used as the target for precipitation only events. Sea level rise would increase both these target water levels. Since the sea level rise has not yet occurred to apply sea level rise to the target elevations, would yield less conservative results during the early life of the project. Accordingly, the current sea level was used to set water surface targets to provide values that are both reasonable and conservative.
- Pump station tail water – Increases in sea level would create additional tail water on the pump discharges. Since the intent is for the pumps to lift the flow over the flood barrier and sea level is factored into the barrier height it is incorporated into the pumping tail water.

By accounting for projected sea level rise in the manner above a conservative yet reasonable treatment of sea level rise is incorporated into the analysis.

4.3 Existing condition runs

The 24-hour rainfall intensities summarized in Table 5 were simulated under existing conditions for all three project sites. The existing conditions simulations provide a baseline for expected maximum water levels under extreme events, and these maximum water levels were used to gage the performance of the proposed structures. One primary goal of the project is to avoid introducing any potential negative impact to drainage within the design conditions.

For the Existing Conditions simulations, the rainfall intensity was applied uniformly over the watershed, all existing gates were opened, and the tailwater was held to a constant stage corresponding to MHW. Given the times of concentration for the overall watersheds, to account for the full hydrograph, the models simulated a 3-day period, although the storm duration was only 24-hours. After 3 days, peak flows and water surfaces had passed, and the drainage systems were progressing toward normal dry weather conditions.

4.4 Facilities development runs

There were two purposes of these model runs, the first was to estimate set the minimum facilities required to meet the project design conditions. The second was to estimate additional facilities to avoid increasing damages for likely events other than design conditions. Based on the assumed independence of peak rainfall and peak surge events described in Section 3.3, the general development of the structure gates and pumps is based on the following design criteria:

- In consultation with the USACE the 100-year surge event for this analysis was taken as coincident with a 25-year rainfall event; during this event the navigation gates are closed and the maximum water level upstream of the gate shall not exceed MHW. This criterion governs the sizing of the pump. A sensitivity was performed on Clear Creek and it was determined that allowing higher interior water surfaces similar to the 25-year fluvial water surface did not significantly alter the required pumping rate. Slightly different criteria were applied to Offatts Bayou where there was a clear elevation that contained the bayou and where the additional storage provided does have a meaningful impact on the pumping rate.
- The 100-year rainfall event is not coincident with storm surge; during this event the navigation gates are open, the pumps are operational, and the maximum water level shall not exceed the maximum water level from existing conditions, when the 100-year

hydrograph is run against a tail water of MHW. This criterion governs the design of the main navigation gate and potential lift gates.

The resulting gate and pump configuration was subjected to additional performance testing to ensure acceptable performance under a variety of scenarios.

4.5 Additional Facilities Performance Runs

The additional facilities runs were performed to estimate the required facility design if the required level of protection against a precipitation event coincident with a 100-year surge event was changed. The proposed gate and pump configuration developed in Section 4.4 were simulated for the 10, 25, 50, 100 and 500-year rainfall events for the following operational conditions:

- Gates closed, pumps on
- Gates open, pumps on
- Gates open, pumps off

Each of these conditions was compared with existing conditions to gage performance. For the first two conditions, the maximum water level should not exceed that of the existing conditions for a less than 100-year event. The third condition (gates open, pumps off), is used to gage the potential risk to drainage should the pumps fail and to generally understand the sensitivity of the water level to the influence of the gates.

The rainfall event from recent Hurricane Harvey was also simulated for existing and proposed conditions to provide context for the potential impacts of the proposed structure. The timeseries of hourly rainfall intensity recorded at Harris County Flood Control District gage 105 (Mary's Creek at Winding Road) was used to force the coupled hydrology and hydraulic model for each watershed and is shown in Figure 4. The total rainfall recorded was 50.0 inches with a maximum 24-hour 21.9 inches. For conservatism, the rainfall intensity was applied uniformly over each watershed. The storm surge water surface elevations at Galveston Bay were not included in the analysis instead, MHW was used as a boundary condition.

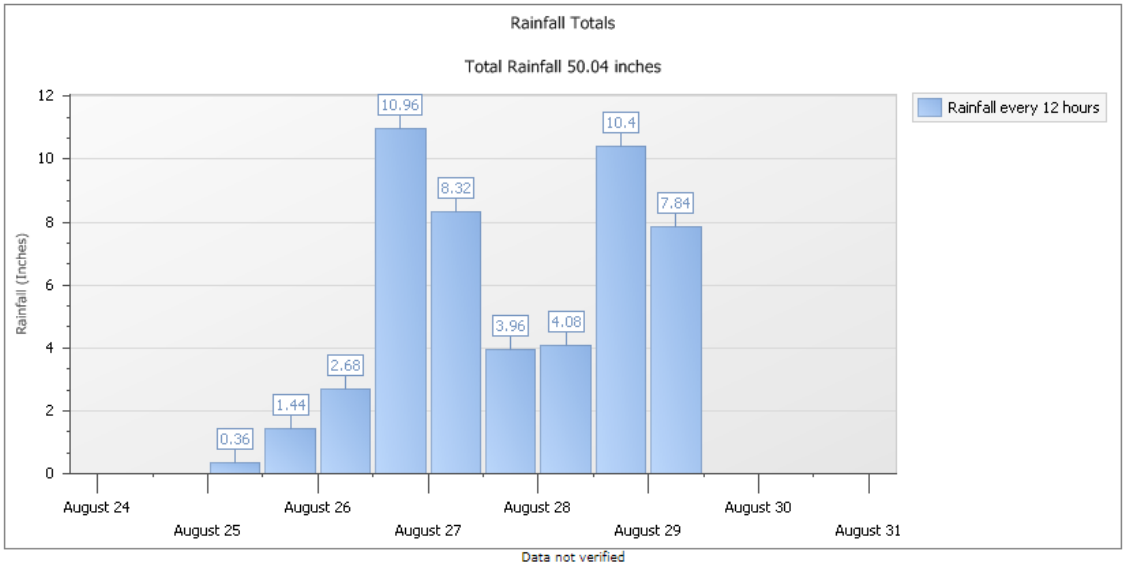

Figure 4. Timeseries of rainfall intensity during Hurricane Harvey at HCFCD Gage 105.

In addition to evaluation of performance under extreme events, the proposed project must also have no adverse impacts to tidal circulation under normal tide and river conditions. Changes to the daily level of tidal inundation can impact the ecology as different species thrive under differing levels of salinity. The proposed flood barriers restrict the hydraulic connection between the interior area and the bay to the open area provided by the navigation gates, which would typically be open. If the gate cannot closely match the performance of the existing channel, there may be adverse impacts to the upstream ecosystem.

Simulations were performed based on a four-day duration of ordinary predicted diurnal tides. The predicted tides were extracted from NOAA-8771013 (Eagle Point) for the Clear Creek and Dickinson Bayou sites and from NOAA-8771486 (Galveston Bridge) for the Galveston site. The time from March 24 – March 27, 2018 was selected for simulation as this period experienced regular diurnal tides roughly ranging between MLLW and MHHW as shown in Figure 5. During future phases of the project, at minimum, 2D circulation modeling should be conducted to investigate any tidal impacts resulting from gate construction.

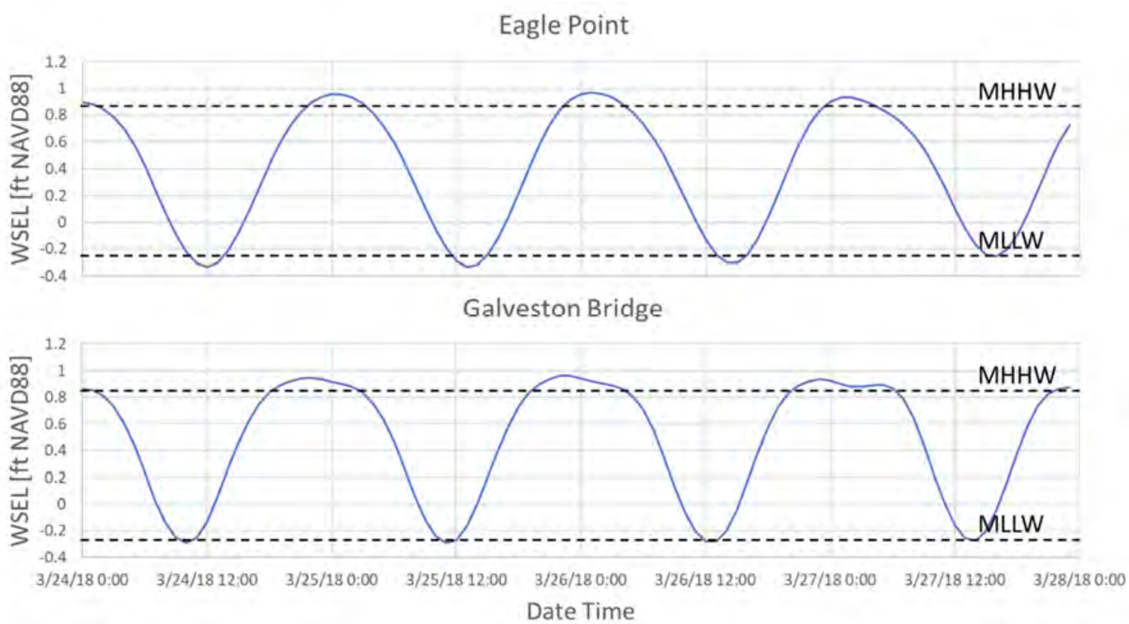

Figure 5. Representative tide signal for simulating impact of structures on tidal circulation.

4.6 Evaluation Runs Considered but screened from analysis

Initially, a set of simulations was proposed to address the potential simultaneous coincidence of both extreme surge and extreme rainfall events. The intent was to see if the facilities could be reduced by taking advantage of periods when the interior water surface was higher than the exterior surface. This would represent the operation of equipping the lift gates with tide gates and controlling the operation of the navigation gates.

These runs would have consisted of running the proposed gates and pumping facilities against a representative 100-year storm surge from the suite of 20 JPM-OS synthetic storms provided by USACE. The intent was to vary the coincidence between the peak surge and start of the24-hour rainfall in 4-hour increments until the flood hydrograph had receded. At the same time real time controls would be applied to open the gates when gravity discharge could occur. However, based on the analysis in Section 3.3, and the additional operational risk they create, these simulations were screened from additional analysis.

4.7 Evaluation Runs Summary

A matrix was developed to summarize all the runs performed for Clear Creek, Dickinson Bayou, and the City of Galveston, as well as the purpose of each model run. This matrix is provided in Table 6.

Table 6. Summary matrix of the total model runs performed for each of the three sites.

Run No.	Model configuration	Rainfall Event	Purpose of Run
1	Existing conditions	10-yr+30%	Evaluate flooding from rainfall events during existing conditions; results are compared to proposed conditions to ensure proposed facilities are not causing negative impacts
2		25-yr+30%	
3		50-yr+30%	
4		100-yr+30%	
5		500-yr+30%	
6	Facility design development with proposed gates closed and pumps on	25-yr+30%	Develop the design facility sizes for the 25yr+30% rainfall
7	Proposed design facilities: gates closed, pumps on	10-yr+30%	Evaluate potential impact from the remaining rainfall events with the design 25-yr+30% facilities in place
8		50-yr+30%	
9		100-yr+30%	
10		500-yr+30%	
11	Proposed design facilities: gates open, pumps on, MHW boundary condition	10-yr+30%	Evaluate potential impact of each rainfall event with the design 25-yr+30% facilities in place, but with the proposed gates in the open position
12		25-yr+30%	
13		50-yr+30%	
14		100-yr+30%	
15		500-yr+30%	
16	Proposed design facilities: gates open, pumps off; MHW boundary condition	10-yr+30%	Evaluate potential impact of each rainfall event with the design 25-yr+30% facilities in place, but with the proposed gates in the open position and the proposed pumps off
17		25-yr+30%	
18		50-yr+30%	
19		100-yr+30%	
20		500-yr+30%	
21	Proposed design conditions: gates open, tidal run	N/A	Evaluate impact from the proposed flood barrier structure on the tidal range inside the water bodies
22	Proposed design conditions: gates closed, pumps on	Harvey	Evaluate impact from Harvey rainfall with design 25-yr+30% facilities in place. ***To be provided in Final Draft***
23	Alternative facility design development with gates closed, pumps on	10-yr+0%	Investigate the required facility sizes for the full range of rainfall events; results are used to develop a rainfall vs. pump size curve
24		10-yr+15%	
25		10-yr+30%	
26		25-yr+0%	
27		25-yr+15%	
28		50-yr+0%	
29		50-yr+15%	
30		50-yr+30%	
31		100-yr+0%	
32		100-yr+15%	
33		100-yr +30%	
34		500-yr+0%	
35		500-yr+15%	

Run No.	Model configuration	Rainfall Event	Purpose of Run
36		500-yr+30%	

5 Clear Creek Watershed

The Clear Creek and Armand Bayou watersheds (collectively referred to as Clear Creek) cover approximately 260 square miles located in southern Harris County and some sections of Galveston, Brazoria and Fort Bend counties as shown in Figure 6. The Clear Creek watershed drains to the east and outfalls into Galveston Bay, while the Armand Bayou drains to the south and connects to the Clear Creek watershed at Clear Lake. The Clear Creek watershed has an average development percentage of about 30%, with most of the development on the downstream portion, while the Armand Bayou watershed has a slightly higher development percentage of 45% with an even distribution of development. Undeveloped areas in both watersheds are mostly covered by pastures that tend to pond during extreme rainfall events (Topical Storm Allison Recovery Project (TSARP) 2004). The boundaries of the Clear Creek and Armand Bayou watersheds are shown in Figure 7.

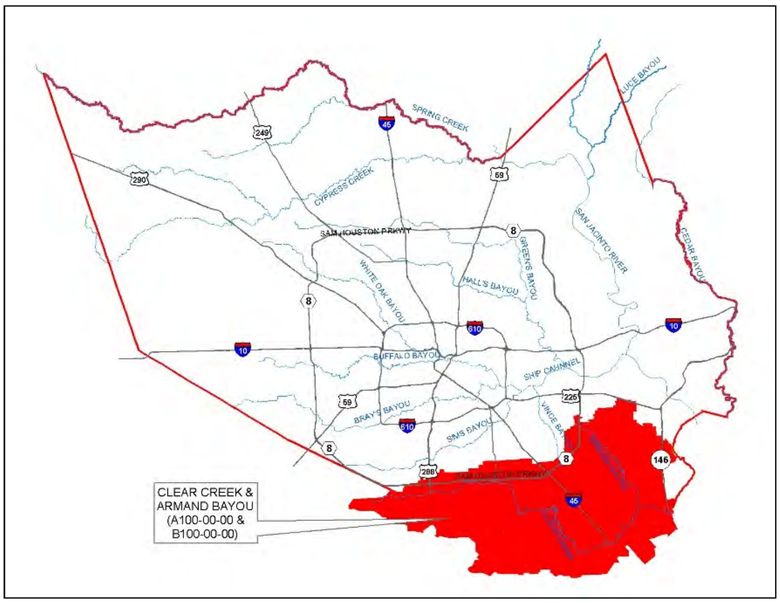

Figure 6. Location map of Clear Creek Watershed (TSARP 2004).

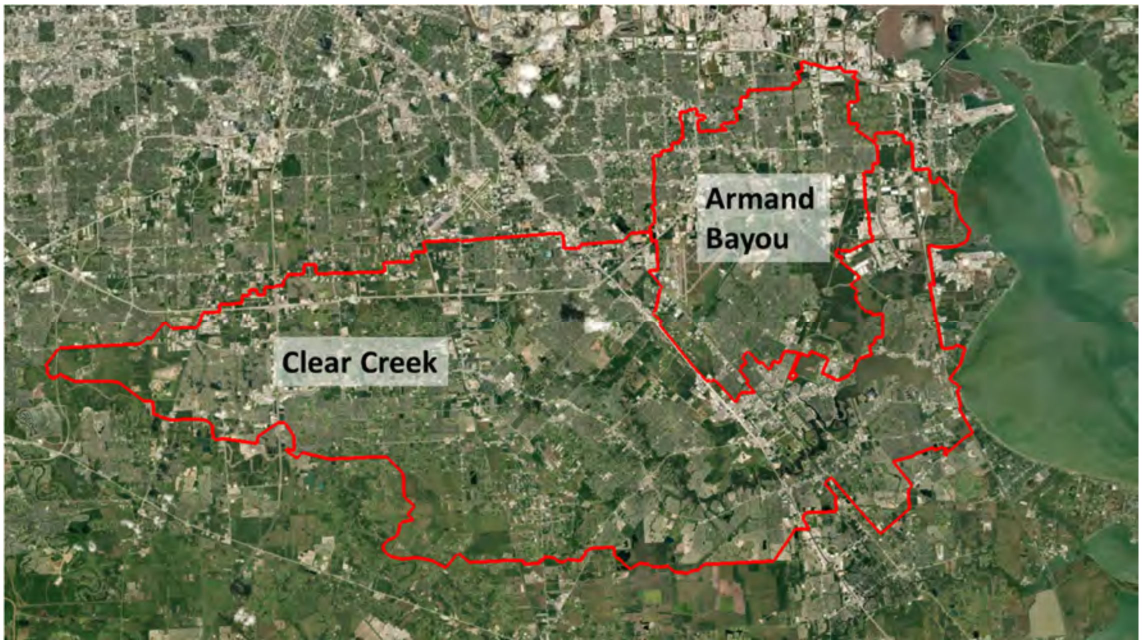

Figure 7. Boundaries of Clear Creek and Armand Bayou watersheds.

The Clear Creek watershed is connected to Galveston Bay at approximately the HWY 146 bridge over Clear Lake. Clear Lake is tidally influenced with a microtidal diurnal tide range of approximately 1.1 feet. The nearest tide station is NOAA-8771013 at Eagle Point as described in Section 2.2. The tidal elevations at this station relative to NAVD88 is summarized in Table 7.

Table 7. Tidal elevations at NOAA 8771013 Eagle Point, TX

Tidal elevations	ft NAVD88*
MHHW	0.86
MHW	0.82
MSL	0.35
MLW	-0.19
MLLW	-0.24

*Conversion to NAVD88 was obtained from NGS (NGS, 2017)

The project flood protection facilities for the Clear Creek watershed consist of a 17-foot high (NAVD88) seawall spanning Clear Lake approximately 300-feet west of the HWY 146 bridge which ties into a 17-foot flood protection on the north and south sides of the lake as shown in Figure 8. The seawall has a large sector gate with a sill elevation of -12 feet (NAVD88) across the main channel and retains the existing series of six 20’ x 20’ lift gates with sill elevations of -15 feet (NAVD88) across the northern secondary channel.

Figure 8. Proposed structure alignment at Clear Lake.

5.1 Data Collection

Most of the data required for hydrologic and hydraulic modeling of the Clear Creek Watershed was readily available through the HCFCD model and map management tool (M3). The M3 tool is a platform for accessing measured and model data for all of Harris County, including the most recent hydrology and hydraulic models for all watersheds in the county. The initial watershed models were developed as part of the Tropical Storm Allison Recovery Project (TSARP) which was initiated in 2004.

The Clear Creek HEC-HMS hydrologic model was acquired from the M3 tool, and includes the Clear Creek Basin (A100-00-00) and the Armand Bayou Basin (B100-00-00). Furthermore, the HEC-RAS hydraulic models developed for the Flood Insurance Study (FIS) streams of Clear Creek (A100-00-00), Taylor Bayou (A104-00-00), and Armand Bayou (B100-00-00) were also acquired from the M3 tool. HCFCD was contacted to confirm that the models available through the M3 tool are the most recent available models. The M3 site states that their models are run in HEC-HMS 3.3.0 and HEC-RAS 3.0.1, consistent with their model development these are the software versions used for this analysis.

In addition to the working models, the associated TSARP reports detailing the development of the models, and Letters of Map Revision (LOMR) detailing the model updates through 2014 were acquired through personal correspondence with Todd Ward at the HCFCD.

5.2 Hydrology Model Development

The existing hydrology model for Clear Creek is pre-calibrated and well documented, so an effort was made to alter the model as little as possible to prevent adverse effects on the model calibration. Recent LOMRs were included in the model where applicable by the HCFCD and the model has been used for regulatory purposes as recently as 2014, this is documented in the model description that it reflects LOMRs through 2014 with an effective date of 12/26/2014

(LOMR 2008, LOMR 2009, LOMR 2012, LOMR 2013 (1), LOMR 2013 (2), LOMR 2013 (3), LOMR 2014).

The sub-basins of the Clear Creek HEC-HMS model use a Green and Ampt loss method and a Clark Unit Hydrograph transform method to develop a hydrograph for each subbasin from a rainfall event. Both methods rely mostly on physical characteristics of the sub-basins that are independent of the rainfall event such as watercourse length, channel slope, percent land urbanization, percent impervious, among others. However, the Storage Coefficient parameter (R) used in the Clark Unit Hydrograph method depends on the Percent Ponding (DPP), which is the portion of the sub-basin in which runoff is held back from reaching a watercourse due to leveed fields. While the DPP is partially dependent on the physical characteristics of the sub-basin, the HCFCD Hydrology and Hydraulics manual prescribes an adjustment factor of the DPP based on the storm size as shown in Table 8.

Table 8. Ponding Adjustment Factor Equations per Storm Event. DPP is the Percent Ponding and RM is the Ponding Adjustment Factor.

Storm Event	RM
(20%) 5-year	$1.31 \times DPP^{0.214}$
(10%) 10-year	$1.28 \times DPP^{0.199}$
(4%) 25-year	$1.25 \times DPP^{0.171}$
(2%) 50-year	$1.23 \times DPP^{0.153}$
(1%) 100-year	$1.21 \times DPP^{0.132}$
(0.2%) 500-year	$1.17 \times DPP^{0.086}$

These adjustment factor equations provided by HCFCD have no direct dependency on rainfall volume. For example, adding 30% to the 10-year storm (7.8") to adjust for changes in rainfall patterns, would yield 10.1" (7.8" +30%= 10.1"). Which is similar in magnitude to the 25-year storm (9.8"). Since the watershed response is dependent on the rainfall volume, applying the equations based on return period is no longer applicable and it would be more appropriate to adjust the values based on rainfall volume.

The storage coefficients from the existing 10-year, 50-year, 100-year and 500-year basins were correlated to the respective 24-hour rainfall volume for each sub-basin using a least squares exponential regression function of the form:

$$R_n = a(I)^b$$

Where R_n is the Storage Coefficient of sub-basin n, and I is the 24-hour rainfall depth. The average R^2 value for the regression from all sub-basins was 0.947, and the minimum R^2 value was 0.662. An example regression from sub-basin *A100A* (farthest upstream) is shown in Figure 9. The storage coefficients for all sub-basins were adjusted according for the 10-year + 30%, 25-year +30%, and 100-year + 30% rainfall volumes.

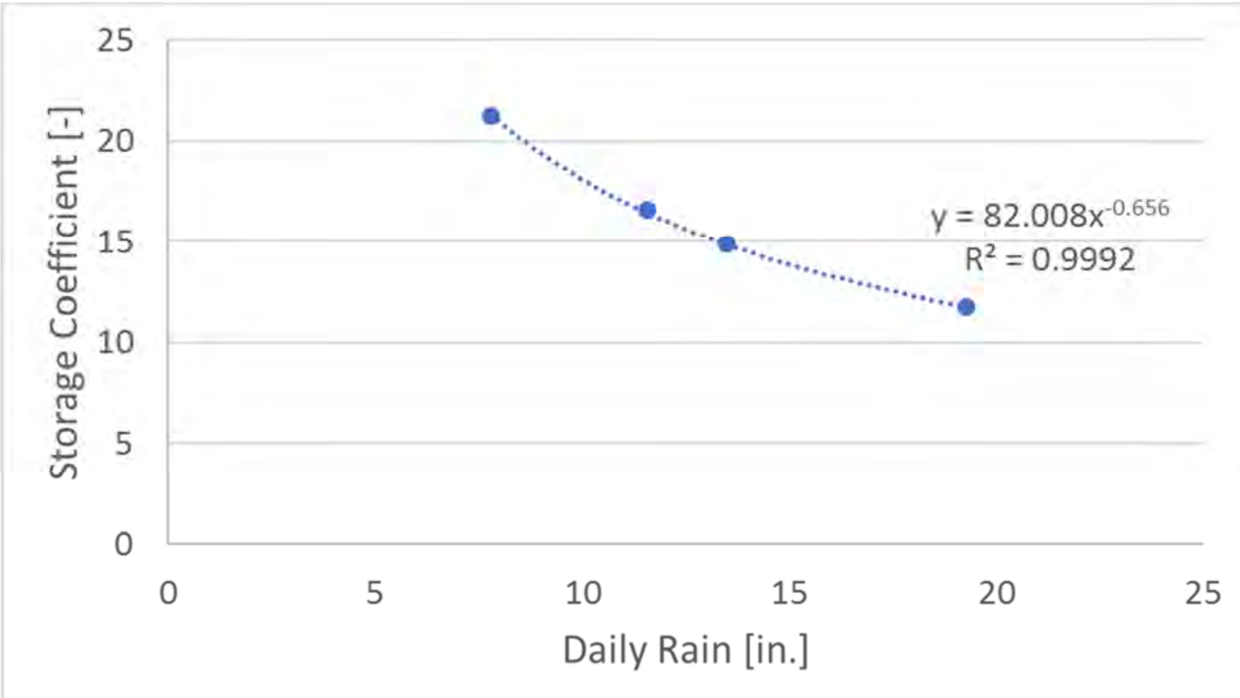

Figure 9. Example Storage Coefficient regression for sub-basin A100A. Filled markers are the existing storage coefficient values and dotted line is the regression function.

To accommodate the 500-year event with the HEC-HMS model, 22 of the storage-discharge tables and 7 of the inflow-diversion tables were extrapolated. The extrapolation method was either linear or polynomial based on optimization of the R^2 value of the regression and is summarized in Appendix C. The final schematic of the HEC-HMS model for the Clear Creek watershed is shown in Figure 10.

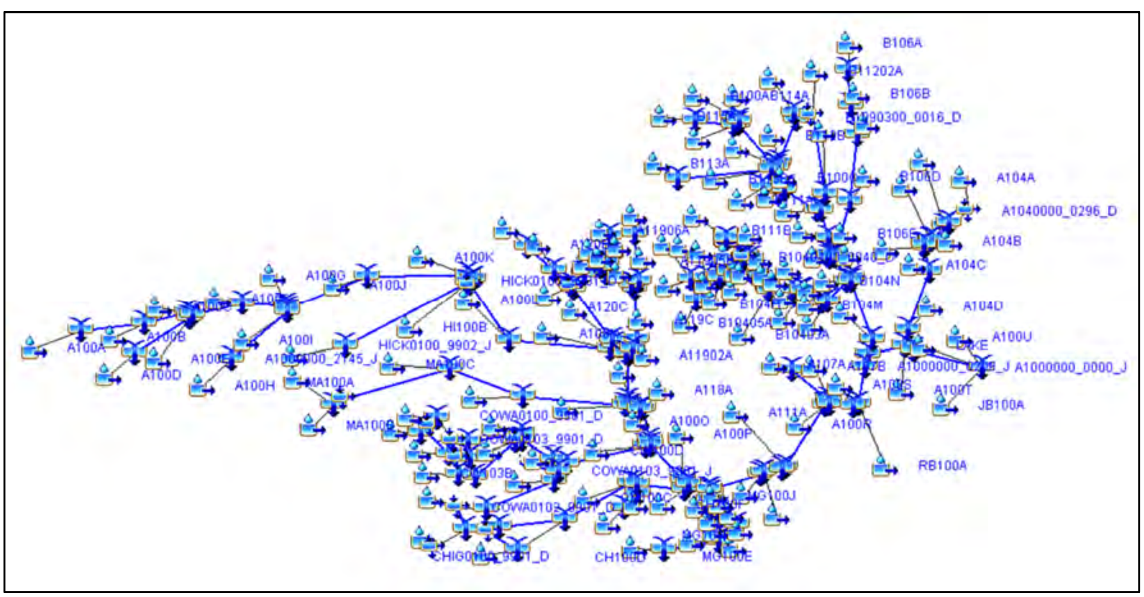

Figure 10. Schematic of HEC-HMS model for Clear Creek.

5.3 Hydraulic Model Development

As with the hydrology model, the existing hydraulic model was also calibrated and well documented, so efforts were made to minimize changes to the model to avoid affecting the calibration (TSARP 2004). However, the model was initially developed to run in steady state which is inadequate for simulating time-dependent tailwater effects due to storm surge or for simulating the operation of pumps Unsteady state HEC-RAS can accept input from HEC-HMS hydrographs transferred between programs using the Data Storage System (DSS). Unsteady state HEC-RAS allows hydraulics to be calculated as well as a more precise representation of the channel routing effects, previously approximated by the Modified Puls method in HEC-HMS. Thus, the HEC-RAS model was modified slightly to run unsteady simulations.

The most significant change was the integration of the Clear Creek (*A100-00-00*), Armand Bayou (*B100-00-00*) and Taylor Bayou (*A104-00-00*) into one cohesive model. This was necessary as Mud Lake and Taylor Lake (in the Armand Bayou and Taylor Bayou basins respectively) provide significant storage capacity for attenuating flows during a rainfall event. The Armand Bayou stream was integrated into the Clear Creek model between Clear Creek River Stations (RS) 18407.6 and 20859.46. The Taylor Bayou stream was integrated into the Clear Creek model between Clear Creek RS 14000.00 and 16112.37.

Additionally, running HEC-RAS unsteady requires a minimum of two unobstructed cross-sections upstream and downstream of any structure. Thus, an additional cross-section was added between the Pearland Parkway Downstream bridge (RS 177429) and the Pearland Parkway Upstream Bridge (RS 177479). The additional section was copied from RS 177454.3 and added at RS 177441.3. The left and right overbank distances of the new section were adjusted to prevent the intersection of adjacent cross-sections.

The boundary conditions for the merged unsteady model were one-way coupled with the HEC-HMS model. The hydrographs from the farthest upstream sub-basins for the Clear Creek, Armand Bayou and Taylor Bayou basins (*A100A*, *B100A*, and *A104A*) were applied as flow hydrographs to the farthest upstream section of each respective stream. The flows from each sub-basin and adjacent tributary were applied as lateral inflow boundary conditions to the River Stations consistent with the junction identified in the model. For Example, sub-basin *A100B* has a downstream junction *A1000000_2385_J* which corresponds to RS 238465.70 in the HEC-RAS model. The tailwater boundary condition was specified as a constant stage value throughout the simulation duration. The final schematic of the HEC-RAS model is shown in Figure 11.

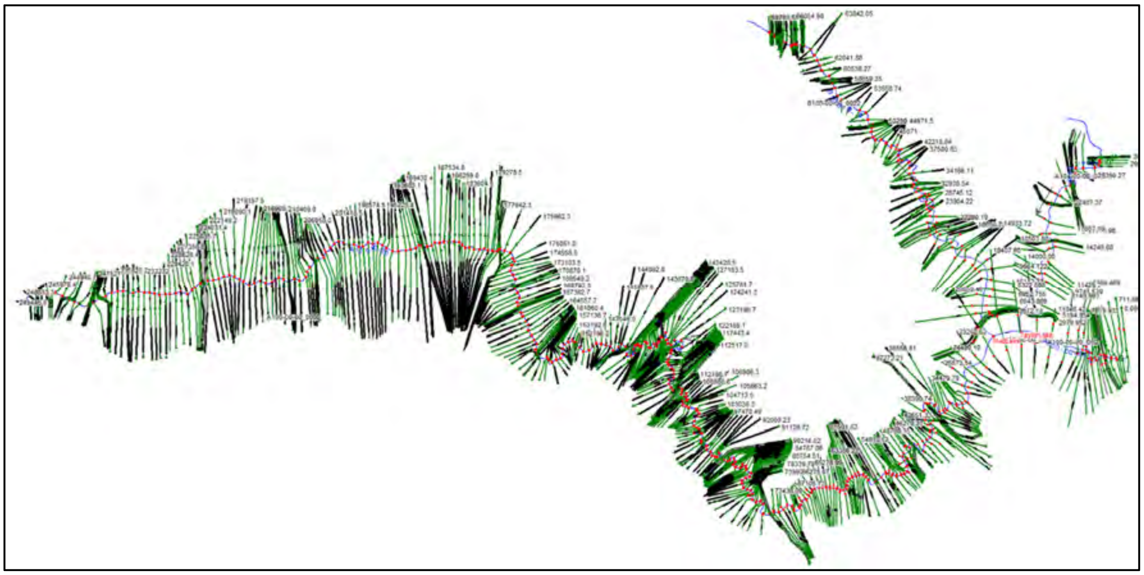

Figure 11. Schematic for HEC-RAS model of Clear Creek.

5.4 Evaluation Scenarios

The evaluation scenarios outlined in Section 4 were analyzed for Clear Creek and are discussed below.

5.4.1 Existing Conditions

As outlined in Section 4.3, the 10, 25, 50, 100, and 500-year (+30%) rainfall events as defined in Table 5 were simulated for Existing Conditions. For each simulation, the peak water level of Clear Lake immediately upstream of the proposed structure location is summarized in Table 9.

Table 9. Peak water level in Clear Lake based on 10, 25, 50, 100 and 500-year return period rainfall events (+30%), tailwater = MHW (0.86 ft NAVD88).

Return Period (+30%)	Peak Water Level [ft NAVD88]
10-year	4.95
25-year	5.68
50-year	6.27
100-year	6.81
500-year	8.52

Note that the peak water level for both the 10 and 25-year events exceed the desired design peak water level of MHW (0.82 ft NAVD88) for the proposed conditions. Thus, the proposed project should improve drainage of Clear Creek for the 10 and 25-year events (+30%). The maximum water level profile for each return period for Existing Conditions is shown in Figure 12.

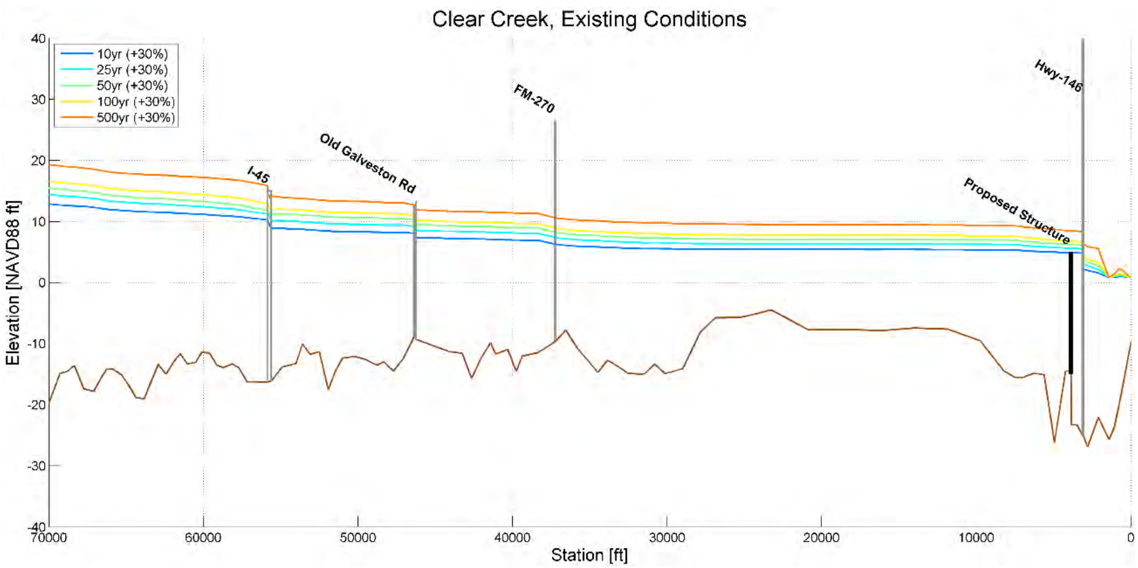

Figure 12. Maximum water level profile by return period at Clear Creek for Existing Conditions.

5.4.2 Facilities Development

For the facilities development runs described in Section 4.4, the tailwater boundary condition at the downstream boundary is held constant at MHW (0.82 ft NAVD88). A preliminary sector gate size was proposed with a 75-foot wide sector gate with a sill elevation of -12 ft NAVD88 centered on the existing channel centerline. The top of the structure elevation was set at +17 ft NAVD88 and connects to the existing gate structure and to the dike on the North and South bank of the lake as shown in Figure 13.

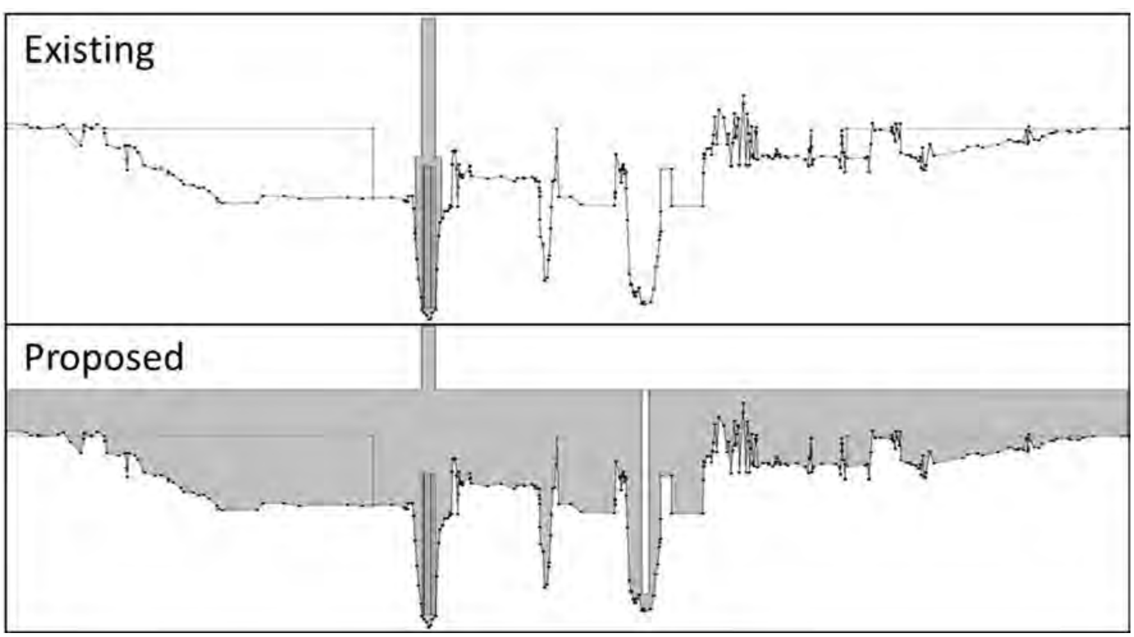

Figure 13. Existing conditions (top) vs. proposed gate alignment (bottom).

As specified in Section 4.4, the design of the pump capacity is based on the 25-year (+30%) event where the gates are assumed to be closed and the water level shall not exceed MHW.

Operationally, a total of four equally sized pumps were assumed, and the water level threshold for initiating each of the pumps was staged by approximately 3-inches as shown in Table 10. The pump on-off levels were staggered to enhance model stability and to reflect realistic operational conditions. Ahead of the storm peak, the water level in the lake is pumped to one-foot below MLLW (-1.24 ft NAVD88) and the pumps are incrementally turned on once the water level of the lake exceeds the pump threshold "on" elevation. Thus, only when the water level exceeds -0.5 ft are all pumps operating at full capacity.

Table 10. Summary of Clear Creek pump operations

Pump	Threshold Elevation – ON [ft NAVD88]	Threshold Elevation – OFF [ft NAVD88]
Pump 1	-1.24	-1.50
Pump 2	-1.00	-1.25
Pump 3	-0.75	-1.00
Pump 4	-0.50	-0.75

Based on the criteria outlined in Section 4.4 and the pump operations outlined in Table 10. A pump capacity of 44,500 cfs is required at Clear Creek. Note that this initial pumping estimate does not include pump capacity to mitigate the design overtopping volumes. See Section 5.5 for a description of how overtopping was incorporated into the pumping requirements. Figure 14 shows a profile comparison of the maximum water level for Existing Conditions and Proposed Conditions. As expected, the proposed conditions show a lower peak water level since the proposed pump capacity is designed to prevent the water level from exceeding MHW.

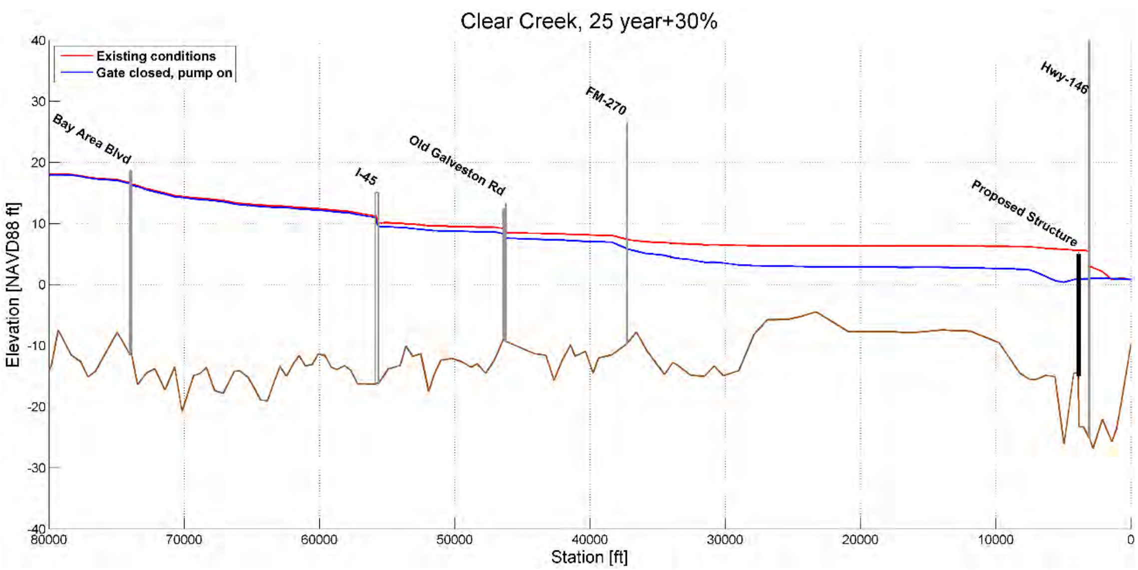

Figure 14. Maximum water level profile comparison of Existing and Proposed Conditions (gates closed, pumps on) for a 25-year event (+30%).

In addition to the 25-year (+30%) rainfall, the required pump capacity to meet the design criteria was also determined for a variety of return period rainfall rates. Figure 15 shows the relationship between 24-hour rainfall depth and required pump capacity. There is a clear linear relationship between 24-hour rainfall and pump capacity for rainfall less than 18 inches. For rainfall greater than 20 inches, the model becomes unstable and yields questionable results.

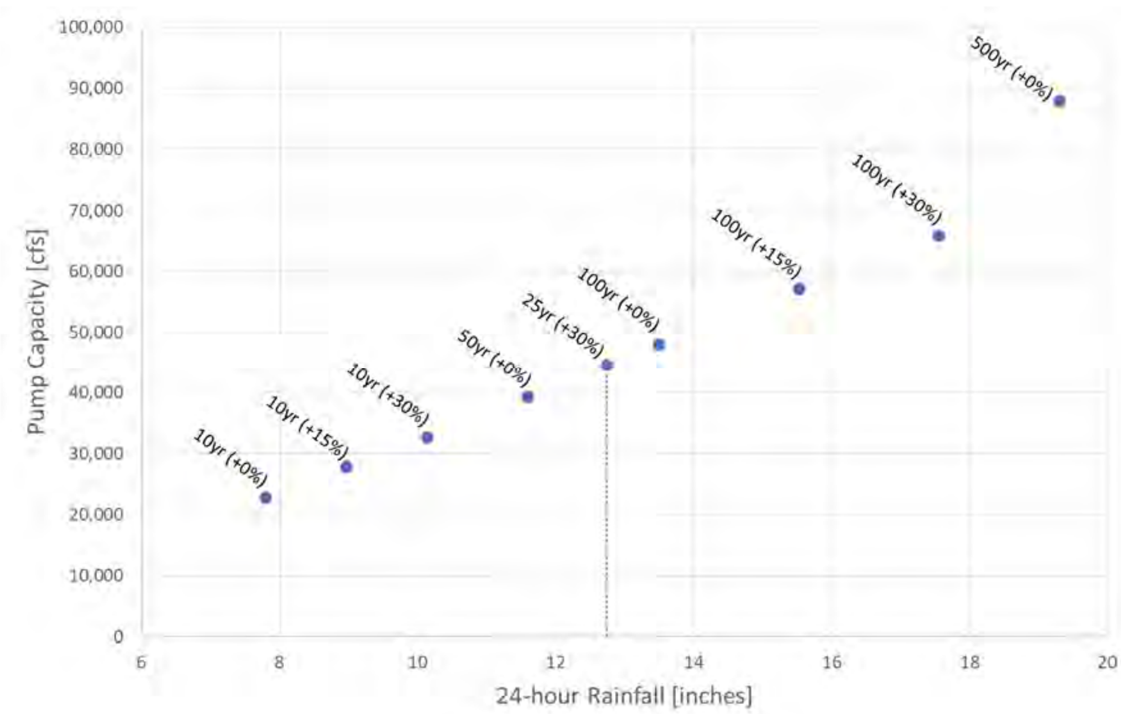

Figure 15. 24-hour rainfall total vs. required pumping capacity for Clear Creek watershed.

As outlined in Section 4.4, the design of the gates is based on the 100-year (+30%) rainfall rate where the navigation gates are open and the design pumps are on. For this scenario, the water level shall not exceed the maximum water level for Existing Conditions under the 100-year (+30%) rainfall rate. The proposed 75-foot sector gate was simulated, and it was found that the maximum water level of Clear Lake was over 2-feet lower than for Existing Conditions. Figure 16 shows a profile comparison of the maximum water level for Existing Conditions and Proposed Conditions. Since the proposed sector gate satisfied the design criteria, there is no need for a wider sector gate or additional lift gates.

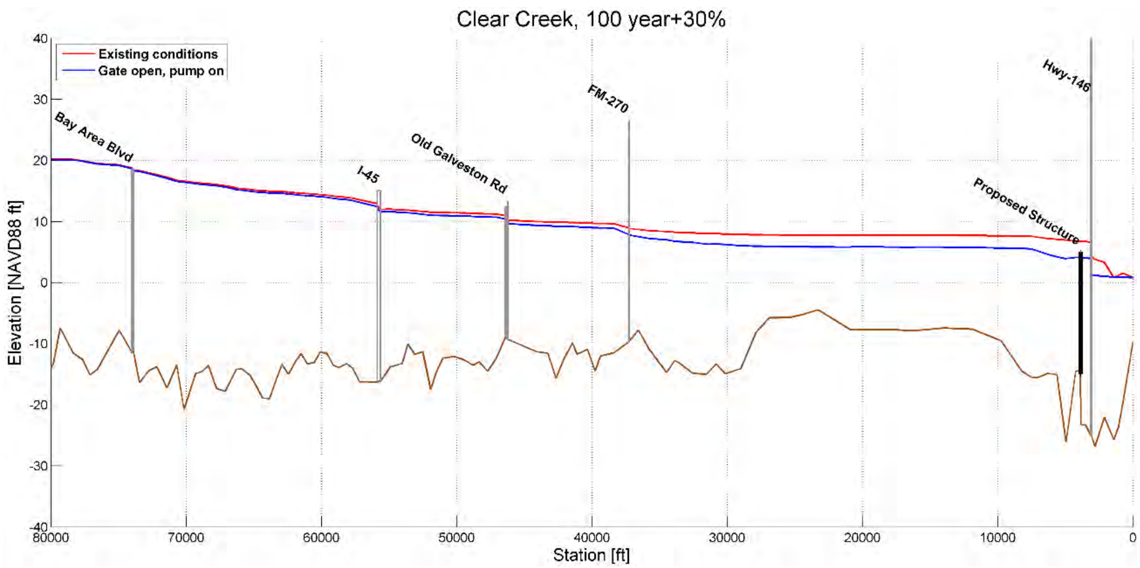

Figure 16. Maximum water level profile comparison of Existing and Proposed Conditions (gates open, pumps on) for a 100-year event (+30%).

5.4.3 Performance Runs

The performance runs outlined in Section 4.5 for the 10 through 500-year return period (+30%) were simulated for the following scenarios:

- Existing Conditions
- Case 1: Gates closed, pumps on
- Case 2: Gates open, pumps on
- Case 3: Gates open, pumps off

The maximum water level for each performance run immediately upstream of the proposed structure are summarized in Figure 17. These show that with the gates open and the pumps on, the proposed facilities will not induce interior flooding and that some level of flood reduction could be expected.

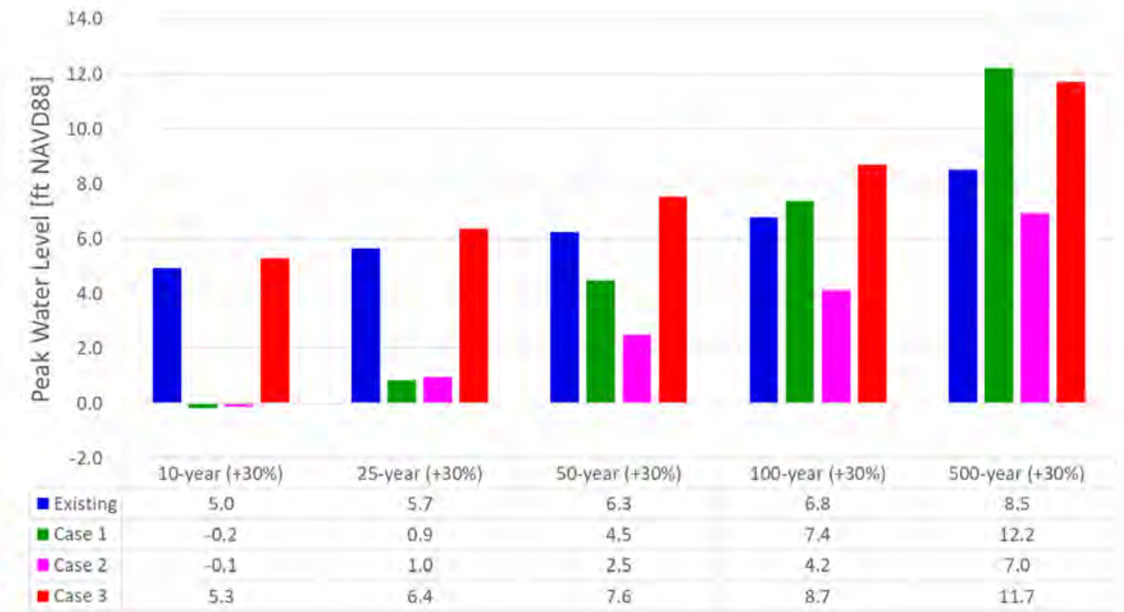

Figure 17. Max water level [ft NAVD88] for performance runs from 10, 25, 50, 100 and 500-year events (+30%) immediately upstream of the proposed structure for Existing Conditions, Case 1 (Gate closed, pumps on), Case 2 (Gate open, pumps on), and Case 3 (Gate open, pumps off).

Finally, as described in Section 4.5, a typical 4-day diurnal tide was simulated for Existing and Proposed Conditions. For this simulation, all gates were assumed to remain open and constant base flows of 100 cfs, 50 cfs, and 50 cfs were assumed at the upstream boundaries of Clear Creek, Armand Bayou, and Taylor Lake respectively. Sensitivity testing was performed on these base flow rates and it was found that an order of magnitude change in the flow rate had negligible impact on the final result. Figure 18 shows a profile comparison of the maximum water level for Existing and Proposed Conditions. Based on this, the structure has a negligible impact on maximum water level under ordinary tide and baseflow conditions.

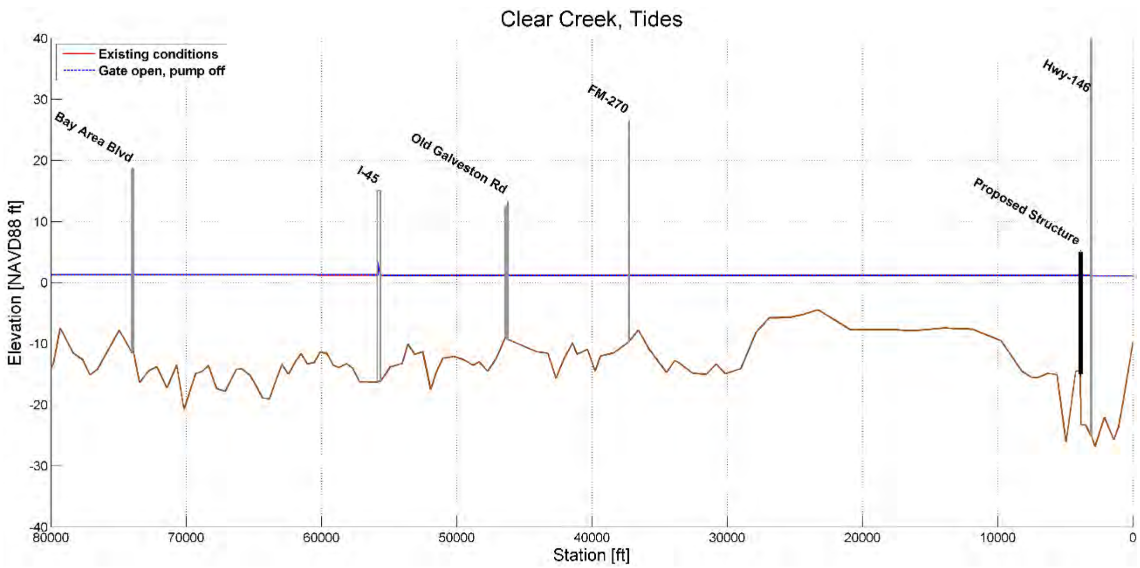

Figure 18. Performance simulation of ordinary tides with baseflow on the Clear Creek watershed.

5.5 Overtopping

An overtopping analysis was conducted at the Clear Creek gate and wall structure to determine if significant overtopping occurs during the 100-year event. A suite of 20 simulation results was provided by the USACE. These storms comprised the JPM-OS suite simulated by the USACE to generate extremal statistics at the project site. The storm with the WSE closest to the 2085, 90% Confidence Interval, 100-year event was selected as the design event for overtopping. Further discussion of the calculation of the extremal WSEs is conducted under the wave loading technical memorandum produced by Mott MacDonald under this contract.

Overtopping was calculated at each timestep in the design storm, using Equation 7.9 and 7.10 of Eurotop, 2016. For this overtopping analysis, the top of the Clear Creek gate and wall structure was assumed to be +17' NAVD88. See Appendix A for a full summary of input conditions and overtopping rates. The overtopping rate calculated at each point in the storm timeseries.

Once a timeseries of overtopping rates was calculated, the overtopping rates were then applied to a representative length of the gate and wall structure, roughly stretching from shoreline to shoreline. The northern edge of the representative length was terminated near the existing Clear Creek Second Outlet structure. North of this point, HWY146 is elevated, and overtopping rates are expected to be significantly lower than those seen along the representative overtopping length. Figure 19 shows the extraction point used for analysis as well as the representative wall length

Figure 19. Model extraction point and representative wall length where overtopping was applied (red). Approximate wall extents also shown (black).

Table 11 shows the peak overtopping rate, representative length, and peak overtopping flowrate experienced at Clear Creek. See Appendix A for a full summary of the timeseries of wave conditions, WSE's, and overtopping rates.

Table 11. Summary of peak overtopping for design event.

Peak Overtopping [cfs/ft]	Applicable Wall Length [ft]	Peak Overtopping Flowrate [cfs]
0.39	2,985	1,161

The peak overtopping flowrate that occurred during the design event was linearly added to the pump capacity calculated in Section 5.4.2. This linear addition assumes that the peak overtopping flowrate and the peak riverine flowrate occur at the same time. Thus the revised pump capacity for Clear Creek is 45,651 cfs. In future stages of the study, a joint probability analysis correlating riverine and overtopping flowrates should be conducted.

5.6 Recommended facilities

Based on the analysis summarized in Section 5.4 and Section 5.5, the proposed 75 foot wide sector gate with sill elevation of -12 ft (NAVD88) supported by a pump station with a capacity of 45,651 cfs meet the design criteria for Clear Creek. The proposed pump capacity is more than twice the capacity of the West Closure Complex in New Orleans, but brings some significant benefits. With these proposed facilities, drainage will be improved for a 10 and 25-year rainfall (+30%), even when coupled with a 100-year storm surge. For a 50, 100 and 500-year rainfall

(+30%), the proposed facilities may also improve drainage if not coupled with storm surge. The proposed facilities show negligible impact to tidal circulation.

5.7 Concept Plans

The reader is referred to the drawing set developed under Task D of this scope of work for conceptual drawings representing the proposed design at Clear Creek.

6 Dickinson Bayou Watershed

The Dickinson Bayou watershed is a coastal basin located in Galveston and Brazoria Counties with a drainage area of 98 sq mi as shown on Figure 20. Dickinson Bayou watershed drains from west to east discharging in Galveston Bay at State Highway 146. Its land use is characterized by a combination of developed areas, farm lands, and undeveloped areas. The elevation in Dickinson Bayou ranges from 15 to 0 ft NAVD88. The nearest tide station is NOAA-8771013 at Eagle Point as described in Section 2.2. The tidal elevations at this station relative to NAVD88 is summarized in Table 7.

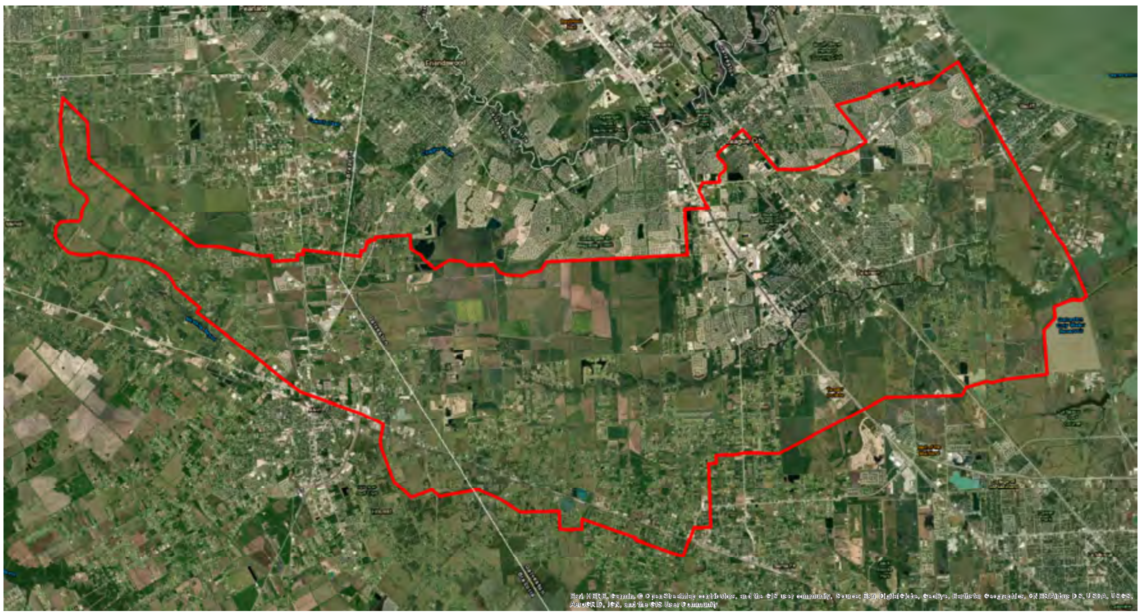

Figure 20. Location Map for Dickinson Bayou Watershed delineated in red

The proposed flood protection facilities for the Dickinson Bayou watershed consist of a 18-foot high (NAVD88) seawall spanning the mouth of Dickinson Bayou from its east to west bank, as shown in Figure 21. The proposed seawall has an original 60 ft wide sector gate with a sill elevation of -9 feet (NAVD88).

Figure 21. Proposed structure alignment at Dickinson Bayou.

6.1 Data Collection

Extensive data collection efforts took place to locate the existing 1991 FEMA FIS hydrology and hydraulic studies (FEMA, 1991). A request was filed in the FEMA Library Services using the FIS Data request form. The data request was assigned to a FEMA case manager and the request was later escalated and expedited. Mott MacDonald was directed to Carrie Hoover, assigned case analyst from Baker. Hoover officially indicated that no existing hydrology data and/or models are available for Dickinson Bayou (Hoover, 2018). For verification purposes, FEMA directed Mott MacDonald to Halff as the consulting firm in charge of FEMA Region 6, which includes Dickinson Bayou. David Patterson from Halff indicated there is no hydrology model available for Dickinson Bayou since the latest FEMA FIS did not include a drainage study (Patterson, 2018). Hence, the data search concluded that no existing hydrology data and/or models are available for Dickinson Bayou. In addition, extensive effort was placed on finding existing stream gage data. The search included contacting universities and consulting firms; no relevant existing stream gage data was found.

Due to the lack of existing hydrology models and stream gage data, a hydrology model was developed for Dickinson Bayou watershed using the Harris County Flood Control District methodology (Harris County Flood Control District, 2009) and limited available input data, which included:

- FEMA peak discharges for 10-, 50-, 100-, and 500-yr events, no associated hydrographs were provided, but were used to validate peak flow results.
- 24-hr rainfall data (Harris County Flood Control District, 2009)
- USGS LiDAR 1/3 arc-second data (USGS, 2017)
- Land use (Texas GLO, ND)
- Aerial photography from ArcMap database

The FEMA Library Request provided hydraulic model HEC2 data from 1979 (Hoover, 2018). The provided data consisted of paper scanned HEC2 model inputs. The data was manually converted to a digital format which was used to build a working HEC-RAS model. The 1979 HEC2 data Included cross-section geometry, bridge location and geometry, Manning's n, and inflow locations.

6.2 Hydrology Model Development

Using the available topographic data (USGS, 2017) and aerial photographs, a drainage map of Dickinson Bayou, shown in Figure 22, was created with a total of 24 sub-watersheds.

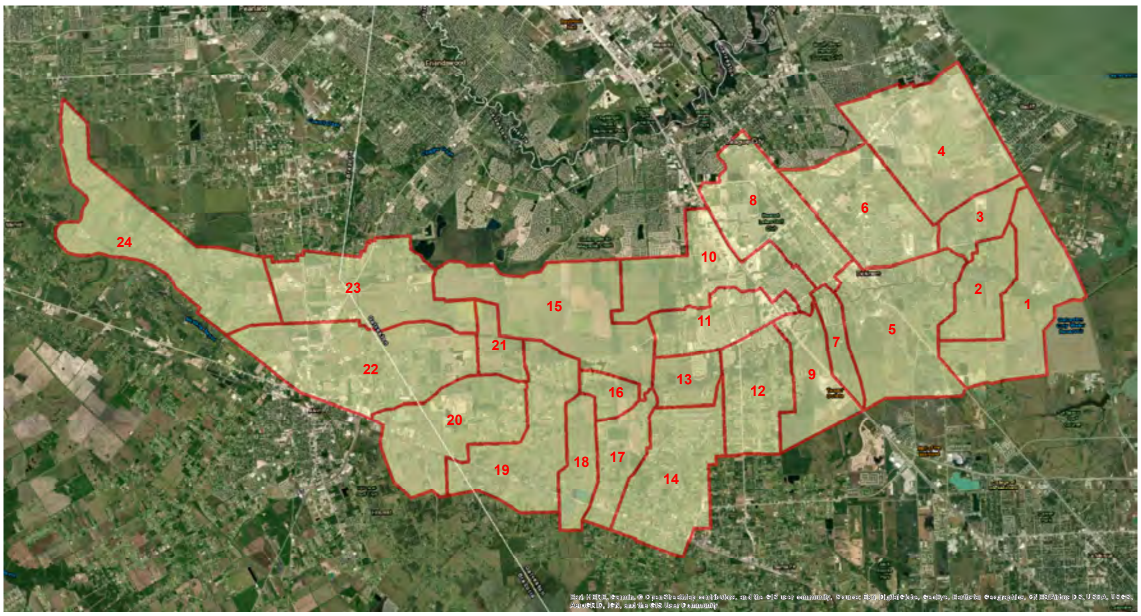

Figure 22. Drainage map of Dickinson Bayou watershed

The Harris County methodology employs the Green and Ampt loss method and the Clark Unit Hydrograph transform to develop a hydrograph for each sub-watershed from a rainfall event. The two main parameters from the unit hydrograph are the time of concentration (TC) and the storage coefficient (R). Both methods rely mostly on physical characteristics of the sub-basins such as watershed length, watershed centroid, watershed centroid, channel slope, percent land urbanization, percent channel improvement, percent channel conveyance, percent ponding, and onsite detention. Such parameters were extracted based on available data and visual inspection of aerial images; hence, due to the limited data available, the following assumptions were made in this conceptual level study:

- Sub-watersheds were delineated primarily based on the contours built from the available topographic data (USGS, 2017). Google Earth images were also employed in delineating the sub-watersheds. No detailed field work took place for this level of effort. However, the notes and observation from the reconnaissance field trip performed on April 2018 were taken into consideration while delineating the sub-watersheds. Following the combination of topographic data, field observations, and Google Earth images, the presence of aqueducts, impounded channels, flow splits, and levees were taken into account when creating the hydrology model. A detailed field campaign for refining and confirming flow patterns and sub-watersheds boundaries is recommended for the next level of effort for Dickinson Bayou hydrology study.

- Percent land urbanization (urbanized areas) was delineated using ArcMap database aerial images which are assumed to be representative of the current development in Dickinson Bayou.
- Percent channel improvement was delineated using ArcMap database aerial images which are assumed to be representative of the current channel improvements, such as lining or impounding.
- Percent channel conveyance was estimated by means of a linear regression assuming maximum conveyance of 30% in the most developed sub-watersheds versus 100% conveyance in undeveloped sub-watersheds.
- Percent ponding was assumed to be the sum of the farm land and swamp areas minus the conveyance areas. Such values were extracted from the land use data base.
- Onsite detention was delineated using ArcMap database aerial images which are assumed to be representative of the current development in Dickinson Bayou.

The time of concentration and storage coefficients were calculated based on the Harris County methodology (Harris County Flood Control District, 2009); when necessary, the storage coefficient was adjusted based on the percent ponding and the storm event return period based on the storm size as shown in Table 8. Using the sub-watershed parameters and following the Harris County procedures, a hydrology model was set up in HEC-HMS (USACE, 2016).

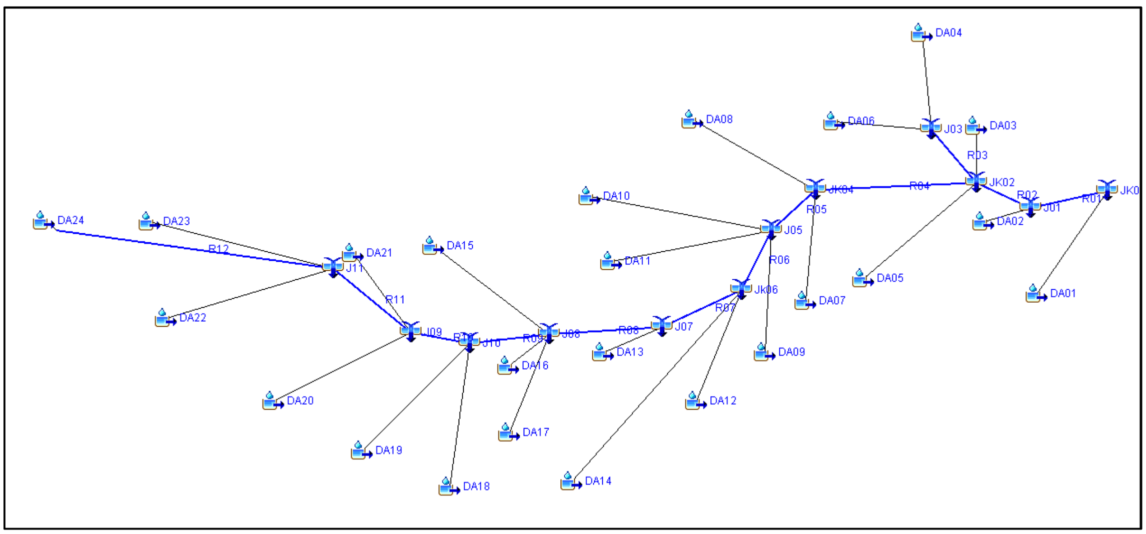

Figure 23. Schematic representation of Dickinson Bayou in HEC-HMS

The HEC-HMS model peak discharges were compared against FEMA's discharge results. The comparison between FEMA and Mott MacDonald (MM) model results, shown in Table 12 with reference to Figure 24, were deemed appropriate for this level of analysis. Note that the 100-year discharge near the project site (see At Route 146 row in Table 12) calculated by the Mott MacDonald model compares very well with the FEMA results. Even though the FIS FEMA results are dated 1991, the actual hydrology study was conducted in 1979 (Patterson, 2018). It is thought that differences in peak flow at the upstream reaches are attributable to the flowing:

- Differences in the delineation of sub-area between the two models. Mott MacDonald upper reach consists of 53.9 sq mi whereas FEMA consists of 43.0 sq mi. The increase in drainage area is directly proportional to the runoff; hence, Mott MacDonald exhibits higher discharge at Interstate 75.
- Possible changes in time of concentration paths, which would tend to manifest themselves in a more pronounced way in the upstream reaches.

- Possible revisions to the sub-area boundaries due to grading and stormwater features, particularly given the flat terrain.
- Differences in locations where flow from sub-areas was introduced into the main channel.
- Upon visual inspection of chronological aerial images of Dickinson Bayou watershed it was noted significant urban development has been taking place since 1979, particularly in Gum and Benson bayou sub-watersheds. Increased development leads to higher runoff which might be associated with increasing discharge from 1979 to 2018 at Benson and Gum Bayou.
- It is assumed the FEMA FIS (1991) study was conducted using HEC1 whereas the present study follows the methodology from Harris County (Harris County Flood Control District, 2009). Different methodologies employ different parameters for calculating runoff which can potentially lead to differences in results.

Since the project study area is at the downstream end, agreement between the current model and FEMA published data in the area was considered to be the more important validation of the model's performance.

Table 12. Comparison between Mott MacDonald (MM) and FEMA discharges for Dickinson Bayou watershed

Dickinson Bayou	Drainage Area FEMA [sq mi]	Drainage Area MM [sq mi]	Q 100yr FEMA [cfs]	Q 100yr MM [cfs]
At Interstate 75 (I-45)	43	53.9	5,920	11,760
At Confluence with Benson Bayou	70	70.6	12,000	16,497
At Confluence with Gum Bayou	89	90.7	17,100	20,798
At Route 146	99	98.3	22,000	22,331

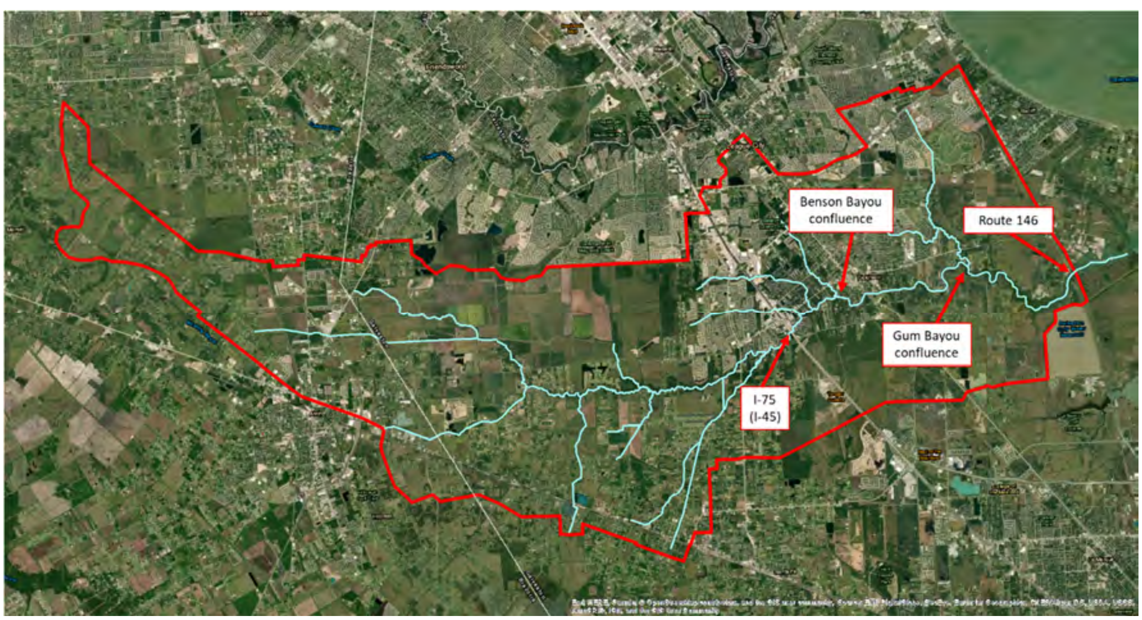

Figure 24. Location map for comparison between Mott MacDonald and FEMA discharges for Dickinson Bayou watershed

For this level of analysis, agreement between the current model and FEMA published data in at the project site was considered sufficient. For future analysis, it is recommended to install streamflow gages at HWY 146 and at Cemetery Rd, sub-basins 1 and 13, respectively (see Figure 22. HWY 146 represents the total drainage area of Dickinson Bayou. Cemetery Rd represent three criteria: (1) the point of largest different between Mott MacDonald and FEMA results, (2) the location on Dickinson Bayou where the general percent land urbanization changes from rural to developed, (3) the upstream limit of the HEC-RAS model (see Section 6.3).

The newly acquired streamflow data in combination with precipitation data will allow for a more robust calibration of the hydrology model. A more robust hydrologic model will better serve as input to the hydraulic model from which design parameters are extracted.

6.3 Hydraulic Model Development

As mentioned in Section 6.1, HEC2 model data from 1979 was provided by FEMA (Hoover, 2018) in pdf format. The HEC2 model data, including cross-section elevations, bridge geometry, Manning's n, contraction and expansion coefficients, was digitized and converted to a steady state HEC-RAS model version 5.0.5. Therefore, the Dickinson Bayou cross-sectional data and channel profile elevations are not representative of current (2018) conditions at the project site. However, due to the lack of updated data at Dickinson Bayou, the results presented in this report are based on 1979 topographic channel data. The existing conditions referred throughout Section 7 are not representative of 2018 but of 1979. It is highly recommended for the next level of analysis to conduct a topographic data collection campaign with the goal of properly mapping updated channel topography including left bank, main channel, and right bank, at Dickinson Bayou.

The elevation data in the 1979 HEC2 model was assumed to be referenced to the NGVD29 vertical datum. The conversion between NGVD29 and NAVD89 at the project site was found to be 0.01 ft. Such variation was found to be insignificant and is outside the precision of the model since typically elevations are reported to the nearest tenth of a foot. Thus, datum conversion was not necessary for the HEC-RAS model data, and all units reported are in ft NAVD88.

Upon visual inspection of chronological aerial images of Dickinson Bayou watershed it was noted that changes to the 1979 bridge geometry used in the HEC2 study had taken place. Hence, the HEC-RAS model was adjusted to match the existing geometry of the 2018 bridges. It was imperative to represent the bridge geometry as accurate as possible given this level of effort and the lack of data to properly represent the current water surface profiles at Dickinson Bayou for the 2018 conditions. Several reasonable adjustments were made to the 1979 FEMA model primarily based on observation of aerial images; the changes include the following:

- Bridge elevations, including deck and pier, were assumed to be the same as 1979. No bridge geometry data, as-built drawings, field check, or surveys was used to verify the existing bridge elevations. It was assumed that based on modern regulatory compliance, the height of the upgraded and/or new bridges should comply with discharge requirement that would maintain the same or better level of drainage.
- Upon visual inspection of chronological aerial images, it was noted some bridges were found to be relocated with respect to 1979. In such cases the bridge cross-section geometry was shifted to the existing location of the given bridge.
- Bridge widths were updated by measuring the most recently available aerial images to properly account for flow constrictions.
- The number of spans, e.g. north and south bounds, was updated based on the most recently available aerial images. The spacing between the spans was measured using

aerial images; each bridge span represents an independent bridge in the HEC-RAS model. When new bridge spans were added, the closest stream cross-section to the 1979 bridge was assumed to be representative of the cross-section in between the new spans.
- HEC2 pier data were included in the model as internal cross-sections as opposed to using bridge piers. Bridge piers were only used on I-146.

The following bridges and their respective adjustment were included in the HEC-RAS model presented in this study:

- State Highway 146 (station 6201 and 6326): The new State Highway 146 bridge is now a 2-span bridge instead of 1-span.
- Railroad (station 6451): the 1979 bridge model included a railroad bridge; currently the railroad has been abandoned and the bayou crossing has been demolished. The abandoned railroad is the proposed location of the storm surge gates at Dickinson Bayou.
- Rail Road (station 39201): the 1979 railroad crossing is assumed to remain unchanged.
- Route 3/Galveston Road (station 41001): the 1979 bridge has been demolished and the road relocated downstream; the bridge widened from 26 ft to approximately 72 ft.
- I-45 (52301, 52356, 52503): the 1979 bridge has been widened remaining in the same location; the new I-45 bridge is now a 3-span bridge instead of 2-span.
- Route 646 (station 60750): the 1979 bridge has been widened remaining in the same location
- Cemetery Rd (station 69201): the 1979 bridge was demolished, relocated upstream, and widened.

The final Dickinson Bayou model extends from station 4100 located downstream of I-45 to station 69429 located downstream of Cemetery Rd (see Figure 25), a schematic of the HEC-RAS model is shown in Figure 26.

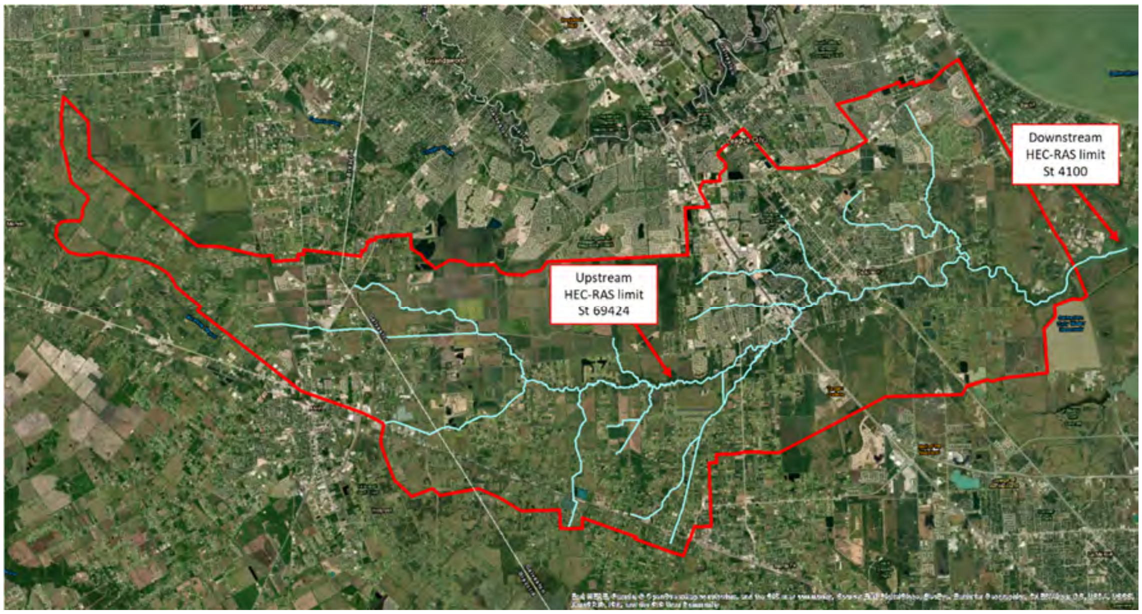

Figure 25. Extents of Dickinson Bayou HEC-RAS model

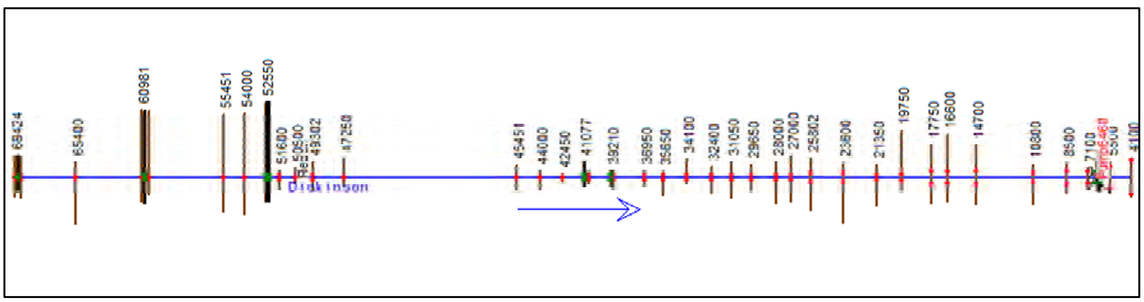

Figure 26. Schematic for HEC-RAS model of Dickinson

Once the 1979 HEC2 model data was adjusted to more accurately represent the existing bridges, a steady state HEC-RAS model was developed. Since no measured flow rate or stage data was found for Dickinson Bayou, the steady state HEC-RAS model was adjusted to match the water surface profiles computed by FEMA FIS (1991) for the 10, 50, 100, and 500-yr return period floods. Several parameters, including expansion and contraction coefficients, ineffective areas, and Manning's n, were adjusted to yield a good comparison. Based on sensitivity analysis, varying the contraction and expansion coefficients, and the ineffective areas did not make a significant impact on the water surface profiles. The best match to the FEMA results was given by lowering Manning's n by 10%. The latter results were deemed appropriate and such changes constituted the final steady state HEC-RAS Dickinson Bayou model. The Mott MacDonald and FEMA comparison results is shown in Figure 27.

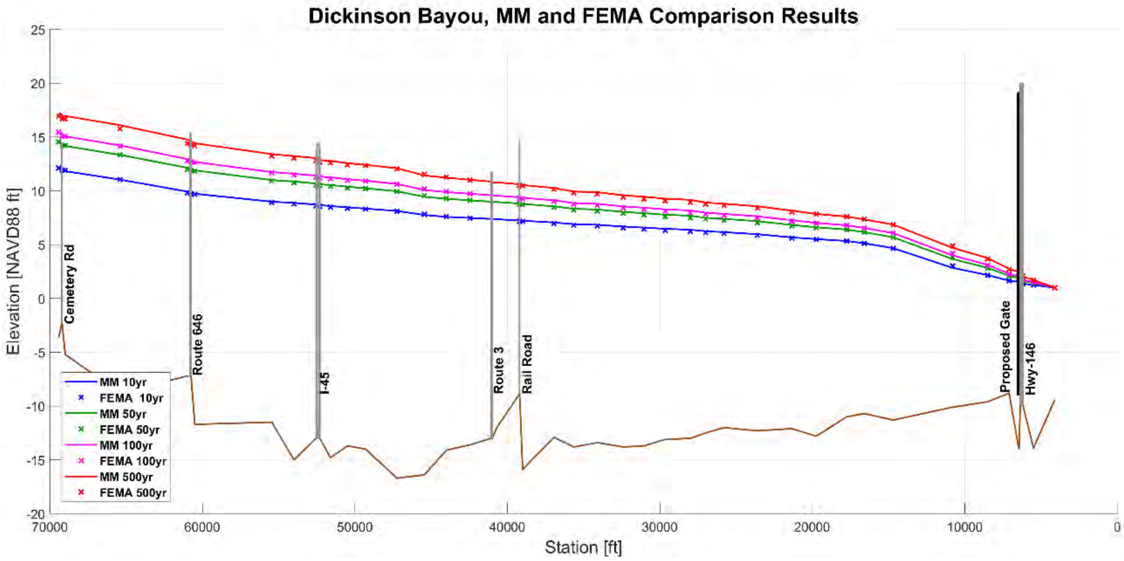

Figure 27. Comparison between Mott MacDonald (MM) and FEMA water surface profiles for Dickinson Bayou

The model was initially developed to run in steady state which is inadequate for simulating time-dependent tailwater effects due to storm surge or for simulating the operation of pumps. Unsteady state HEC-RAS can accept input from inflow hydrographs from HEC-HMS and transferred between programs using the Data Storage System (DSS). Consequently, the steady state HEC-RAS model was converted to an unsteady state model.

The boundary conditions for the unsteady HEC-RAS model were one-way coupled with the HEC-HMS model. The HEC-HMS sub-basins were matched to the corresponding HEC-RAS cross-sections; a total of 14 HMS sub-drainage areas encompassing the totality of Dickinson Bayou watershed were matched. The hydrographs from each of these 14 areas were applied as lateral inflow boundary conditions to the River Stations consistent with the cross-section identified in the model.

6.4 Evaluation Scenarios

The evaluation scenarios outlined in Section 4 were analyzed for Dickinson Bayou and are discussed below.

6.4.1 Existing Conditions

As outlined in Section 4.3, the 10-, 25-, 50-, 100-, and 500-year (+30%) rainfall events as defined in Table 5 were simulated for Existing Conditions. For each simulation, the peak water level of Dickinson Bayou immediately upstream of the proposed structure location is summarized in Table 13.

Table 13. Existing conditions maximum water surface elevation in Dickinson Bayou for 10, 25, 50, 100 and 500-year (+30%) return period rainfall events immediately upstream of the proposed gate location

Return Period	Peak Water Level [ft NAVD88]
10-year (+30%)	1.3
25-year (+30%)	1.7
50-year (+30%)	2.2

Return Period	Peak Water Level [ft NAVD88]
100-year (+30%)	2.4
500-year	4.1

Note that the peak water level for both the 10 and 25-year events exceed the desired design peak water level of MHW (0.82 ft NAVD88) for the proposed conditions. Thus, the proposed project should improve drainage of Dickinson Bayou for the 10 and 25-year events (+30%) immediately upstream of the proposed gate. The maximum water level profile for each return period for Existing Conditions is shown in Figure 28.

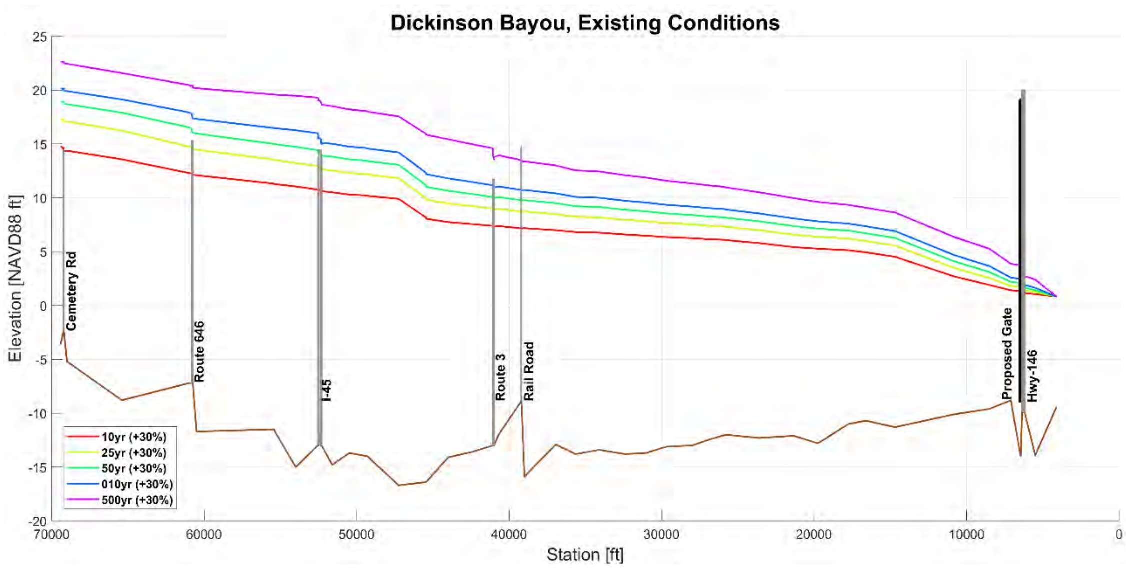

Figure 28. Maximum water level profile by return period at Dickinson Bayou for Existing Conditions.

6.4.2 Facilities Development

For the facilities development runs described in Section 4.4, the tailwater boundary condition at the downstream boundary is held constant at MHW (0.82 ft NAVD88). A preliminary flood wall and sector gate size were proposed. The height of the proposed flood wall was set at 17 ft NAVD88. The preliminary sector gate geometry consisted of 60-foot opening with a sill elevation of -9 ft NAVD88 centered on the existing channel centerline as shown on Figure 29. Unlike Clear Creek, Dickinson Bayou does not have an existing flood gate (see Figure 13).

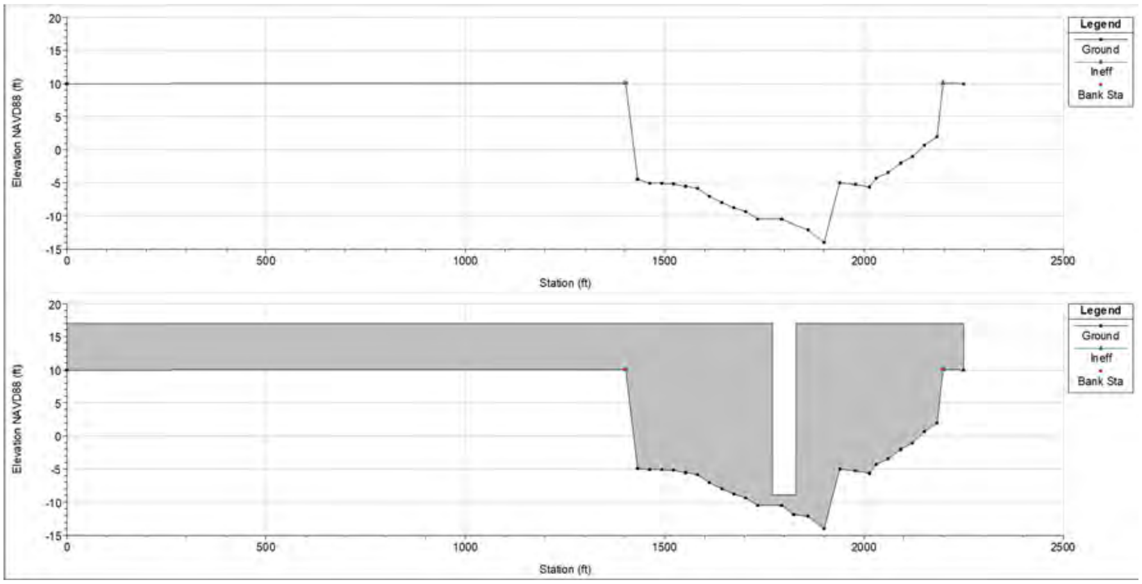

Figure 29. Dickinson Bayou existing conditions (top) and proposed gate alignment (bottom).

As specified in Section 4.4, the design of the pump capacity is based on the 25-year (+30%) event where the gates are assumed to be closed and the water level shall not exceed MHW. Operationally, a total of four pumps were assumed, and the water level threshold for initiating each of the pumps was staggered at 0.25 ft as shown in Table 14. Ahead of the storm peak, the water level in the lake is pumped to one-foot below MLW (-1.19 ft NAVD88) and the pumps are incrementally turned on once the water level of the lake exceeds the pump threshold "on" elevation. Thus, only when the water level exceeds -0.25 ft are all pumps operating at full capacity.

Table 14. Summary of Dickinson Bayou pump operations

Pump	Threshold Elevation – ON [ft NAVD88]	Threshold Elevation – OFF [ft NAVD88]	Pumping rate [cfs]*
Pump 1	-1.00	-1.2	5,000
Pump 2	-0.75	-1.2	5,000
Pump 3	-0.50	-1.2	5,000
Pump 4	-0.25	-1.2	3,700
Total	-	-	18,700

*Note, this rate will be adjusted for overtopping later in this section.

Based on the criteria outlined in Section 4.4 and the pump operations outlined in Table 14. A pump capacity of 18,700 cfs is required at Dickinson Bayou. Note that this initial pumping estimate does not include pump capacity to mitigate the design overtopping volumes. See Section 6.5 for a description of how overtopping was incorporated into the pumping requirements. Figure 30 shows a profile comparison of the maximum water level for Existing Conditions and Proposed Conditions. As expected immediately upstream of the proposed gate location, the proposed conditions show a lower maximum water surface elevation since the proposed pump capacity is designed to prevent the water level from exceeding MHW.

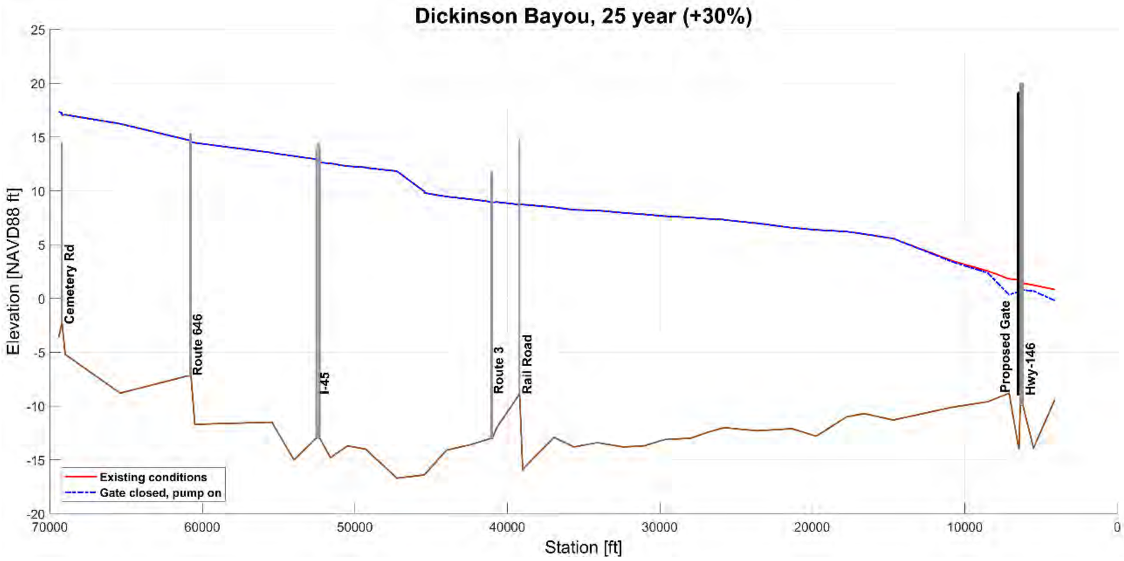

Figure 30. Dickinson Bayou Maximum water level profile comparison of Existing and Proposed Conditions (gates closed, pumps on) for a 25-year event (+30%).

In addition to the 25-year (+30%) rainfall, the required pump capacity to meet the design criteria was also determined for a variety of return period rainfall rates. Figure 37 shows the relationship between 24-hour rainfall rate and required pump capacity. There is a clear linear relationship between 24-hour rainfall and pump capacity. For rainfall totals greater than 22.2 inches, the model becomes unstable and yields questionable results.

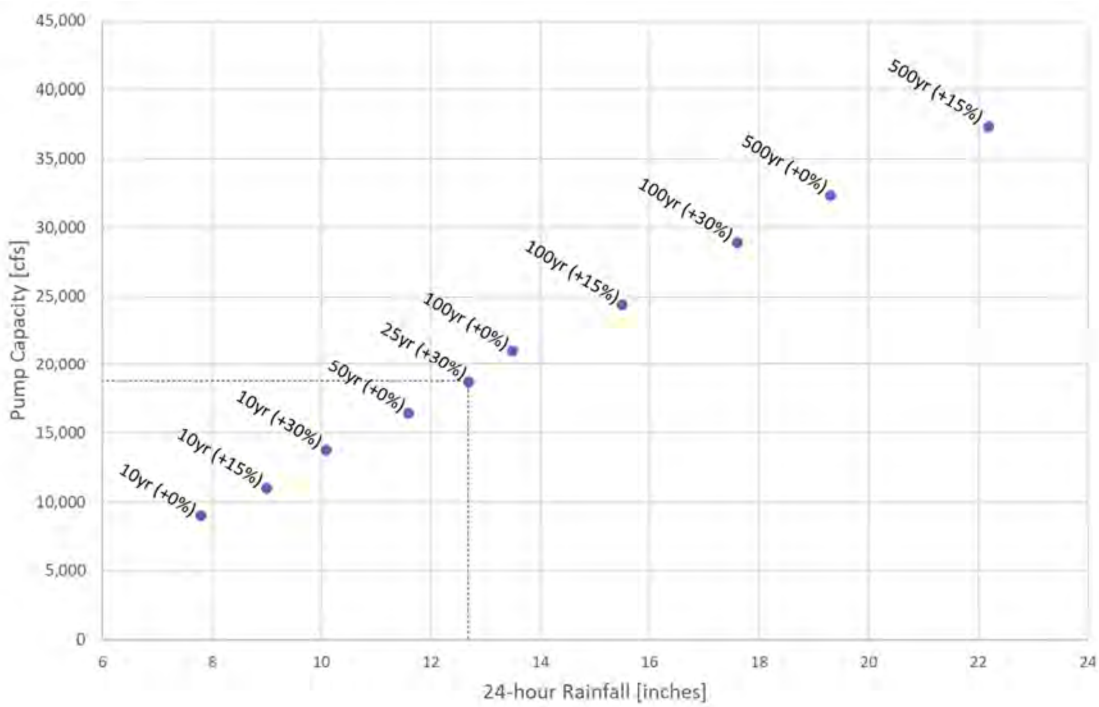

Figure 31. 24-hour rainfall total vs. required pumping capacity (gates closed, pumps on) for Dickinson Bayou watershed

As outlined in Section 4.4, the design of the gates is based on the 100-year (+30%) rainfall rate where the navigation gates are open and the design pumps are on. For this scenario, the water level shall not exceed the maximum water level for Existing Conditions. The proposed 60-foot sector gate was simulated with the 18,700 cfs design pump turned on while the storm surge gate being open. It was found the water surface profile was higher than the Existing Conditions; therefore, it the opening of the gate was widened. Sensitivity testing to different gate opening dimensions was conducted, including 60 ft, 100 ft, and 125 ft, as shown on Figure 32. The optimum gate opening was found to be 100 ft at Dickinson Bayou.

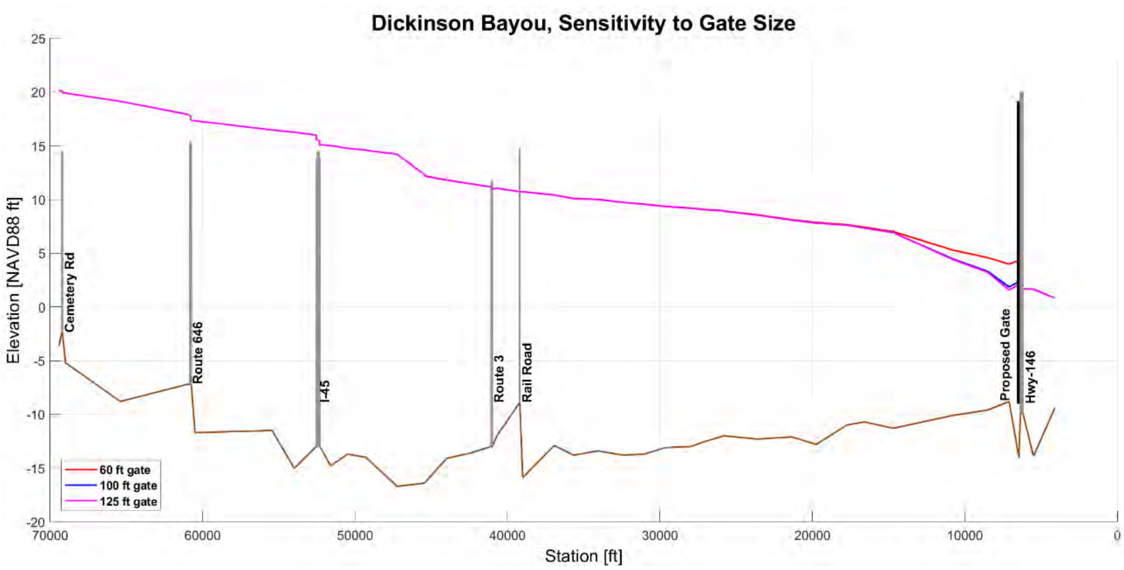

Figure 32. Dickinson Bayou maximum water level profile comparison of Existing and Proposed Conditions (gates open, pumps on) for a 100-year event (+30%) for 60 ft, 100 ft, and 125 ft opening gate.

Figure 33 shows a profile comparison of the maximum water level for Existing Conditions and Proposed Conditions. Since the 100 ft wide sector gate satisfied the design criteria, there is no need for additional lift gates at Dickinson Bayou.

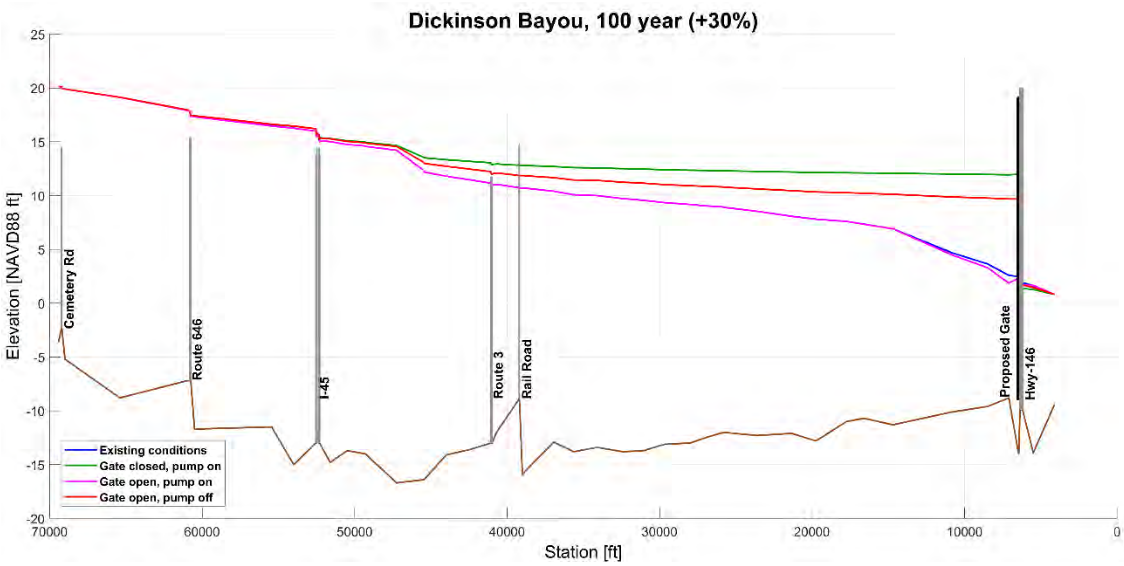

Figure 33. Dickinson Bayou maximum water level profile comparison of Existing and Proposed Conditions (gates open, pumps on) for a 100-year event (+30%).

6.4.3 Performance Runs

The performance runs outlined in Section 4.5 for the 10 through 500-year return period (+30%) were simulated for the following scenarios:

- Existing Conditions
- Case 1: Gates closed, pumps on
- Case 2: Gates open, pumps on
- Case 3: Gates open, pumps off

The maximum water level for each performance run immediately upstream of the proposed structure are summarized in Figure 36. These show that with the gates open and the pumps on, the proposed facilities will not induce interior flooding and that some level of flood reduction could be expected.

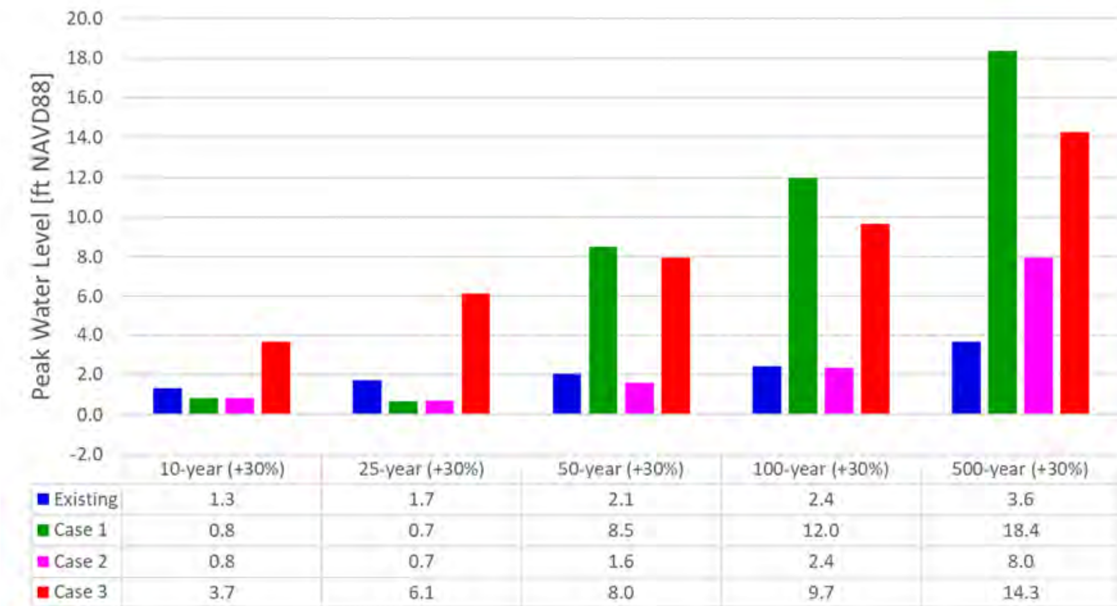

Figure 34. Dickinson Bayou max water level [ft NAVD88] for performance runs from 10, 25, 50, 100 and 500-year events (+30%) immediately upstream of the 100 ft sector gate for Existing Conditions, Case 1 (Gate closed, pumps on), Case 2 (Gate open, pumps on), and Case 3 (Gate open, pumps off).

Finally, as described in Section 4.5, a typical 4-day diurnal tide was simulated for Existing and Proposed Conditions. For this simulation, all gates were assumed to remain open and constant base flows of 100 cfs was assumed at the upstream HEC-RAS cross-section at Dickinson Bayou. Sensitivity testing was performed on the base flow rates and it was found that an order of magnitude change in the baseflow rate had negligible impact on the final result. Figure 36 shows a profile comparison of the maximum water level for Existing and Proposed Conditions. Based on this result, the structure has a negligible impact on maximum water level under ordinary tide and baseflow conditions.

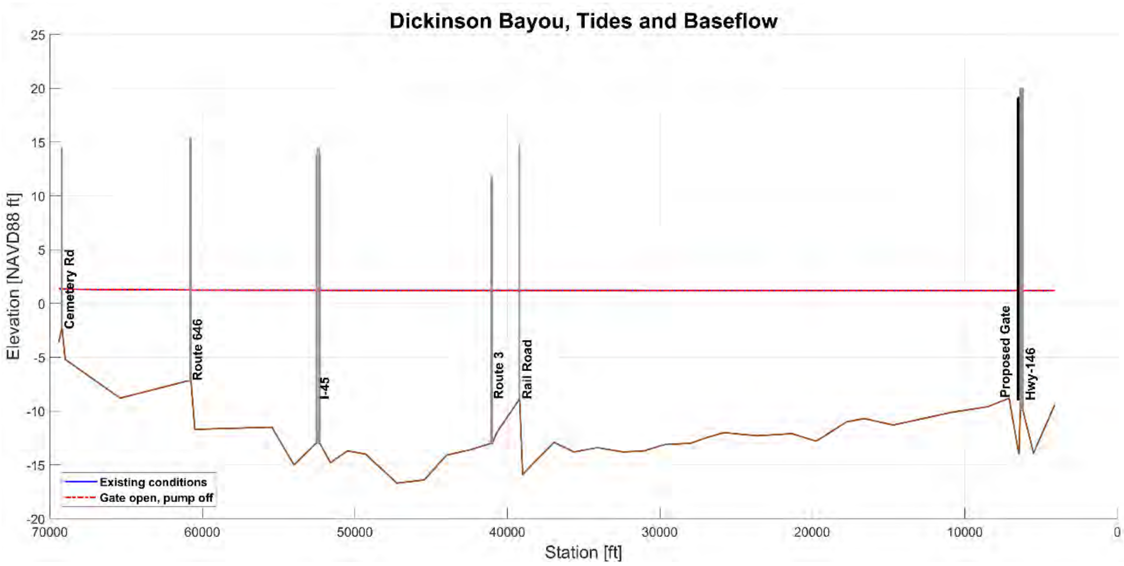

Figure 35. Dickinson Bayou performance simulation of ordinary tides with baseflow

6.5 Overtopping

An overtopping analysis was conducted at the Dickinson Bayou gate and wall structure to determine if significant overtopping occurs during the 100-year event. A suite of 20 storm simulation results was provided by the USACE. These storms comprised the JPM-OS suite simulated by the USACE to generate extremal statistics at the project site. Similar to the Clear Creek analysis, the storm with the WSE closest to the 2085, 90% Confidence Interval, 100-year event was selected as the design event for overtopping. Note that the closest model extraction poin to the gate and wall structure is subject to large fetches across Galveston Bay, which could result in a slightly more energetic wave environment that that experienced at the gate and wall structure. Further discussion of the calculation of the extremal WSEs is conducted under the wave loading technical memorandum produced by Mott MacDonald under this contract.

Overtopping was calculated at each timestep in the design storm, using Equation 7.9 and 7.10 of Eurotop, 2016. For this overtopping analysis, the top of the Dickinson bayou gate and wall structure was assumed to be +18’ NAVD88. See Appendix A for a full summary of input conditions and overtopping rates. The overtopping rate calculated at each point in the

A timeseries of overtopping rates was calculated. The overtopping rates were then applied to a representative length of the gate and wall structure, roughly stretching from shoreline to shoreline. Figure 36 shows the extraction point used for analysis as well as the representative wall length

Figure 36. Model extraction point and representative wall length where overtopping was applied (red). Approximate wall extents also shown (black).

Table 11 shows the peak overtopping rate, representative length, and peak overtopping flowrate experienced at Dickinson bayou. See Appendix A for a full summary of the timeseries of wave conditions, WSE's, and overtopping rates.

Table 15. Summary of peak overtopping for design event.

Peak Overtopping [cfs/ft]	Applicable Wall Length [ft]	Peak Overtopping Flowrate [cfs]
0.48	879	425

The peak overtopping flowrate that occurred during the design event was linearly added to the pump capacity calculated. This linear addition assumes that the peak overtopping flowrate and the peak riverine flowrate occur at the same time. In future stages of the study, the joint probability between riverine and overtopping flowrates should be investigated.

6.6 Recommended Facilities

Based on the analysis summarized in Section 6.4 and 6.5; the proposed 100-ft wide sector gate with sill elevation of -9 ft (NAVD88) supported by a pump station with a capacity of 19,125 cfs meet the design criteria for Dickinson Bayou. The proposed pump capacity mimics the West Closure Complex in New Orleans, LA, which consists of 11 pumps at 1,740 cfs each for a total capacity of 19,140 cfs (USACE, 2014).

With these proposed facilities, drainage will be improved for a 10 and 25 year 24-hr rainfall (+30%), even when coupled with a 100-year storm surge (gate closed and pump on scenario). For a 50 and 100 year 24-hr rainfall (+30%), the proposed facilities may also improve drainage if not coincident with storm surge (gate open, pump on scenario). The proposed storm surge gate and pump complex would cause additional flooding under the 100 year 24-hr rainfall (+30%) event, if the navigation gates are closed due to surge, as discussed earlier this is considered to be improbable. The proposed facilities show negligible impact to tidal circulation.

6.7 Concept Plans

The reader is referred to the drawing set developed under Task D of this scope of work for conceptual drawings representing the proposed design at Dickinson Bayou.

7 Galveston Watershed

This study evaluates the anticipated interior drainage for the City of Galveston within the proposed flood control alignment (Figure 37). For the purpose of this study, the “Galveston Watershed” refers to the area of Galveston that is located within the proposed line of protection alignment. The Galveston Watershed is approximately 8,920 acres (13.9 sq. mi.) in size.

This interior drainage analysis evaluates the quantity of runoff that is produced during a 24-hr extreme rainfall event with the Galveston Watershed. No existing drainage model was available for this study; thus, a basic model was developed for this analysis, utilizing the limited available input data. The interior drainage model for the Galveston Watershed was developed using the Environmental Protection Agency (EPA) Storm Water Management Model (SWMM). SWMM is a dynamic rainfall-runoff simulation model used for single event (or long-term) simulation of runoff quantity from urban areas (USEPA, 2015). For more information on SWMM, refer to USEPA (2015).

The Galveston Watershed is dominated by urban and industrial land coverage. The Watershed also includes the water body of Offatts Bayou.

Figure 37. Figure of the proposed flood control alignment (red line) around Galveston as of November 2017.

7.1 Data Collection and Drainage Map Development

Prior to developing the Galveston Watershed SWMM model, an ArcGIS database was first created to develop a drainage map of the Galveston Watershed. The drainage map was developed using topographic data (USGS, 2018) for the site, which was converted to 1 ft contour lines. The topographic data shows that the areas adjacent to Offatts Bayou flow into the Bayou and the remaindered of the study drainage area drains south to north, from the Gulf side to the bay side. The database was then supplemented with georeferenced aerial imagery.

Using the topographic contours, and the aerial imagery for reference, the Watershed was divided up into sub-basins based anticipated direction of runoff flow. A total of 16 main sub-basins were developed within the Watershed, see Figure 38. shows the drainage map of the Watershed with the sub-basins. Sub-basin nos. 6 – 11 were divided into a and b components to allow simulated runoff to be diverted to separate locations.

Figure 38. Drainage map of the Galveston Watershed, which contains 16 main sub-basins.

Each sub-basin was evaluated and assigned values for the parameters listed below. The following section describes how each parameter was calculated.

- Sub-basin ID
- Area
- Equivalent width
- Average slope
- Average percent impervious area
- Average length

Sub-basin ID – manually assigned; shows the Sub-basin ID in red.

Area – the total area within the sub-basin; calculated using ArcGIS.

Equivalent width – calculated by dividing the area by the average length

Average slope – average slope within the sub-basin; calculated using ArcGIS with the LiDAR contour lines.

Average percent impervious area – average area (in percent) within the sub-basin that is impervious; calculated by assigning "average percent impervious area" values for each land type using the TR-55 Table 2-2a, and averaging those within each sub-basin. Sub-Basin 1 represents the portion of Offatts Bayou within the protected area and since there is no infiltration it was treated as 100% impervious.

Average length – average length of the longest flow path within the sub-basin; measured in ArcGIS

A list of sub-basins and their attributes is provided in Table 16. This data compliments the drainage map for the Watershed (Figure 38).

Table 16. Attributes list for the sub-basins in the Galveston Watershed.

Sub-Basin ID	Area (acres)	Equivalent Width (ft)	Average Slope (%)	Average Percent Impervious Area (%)	Average Length (ft)
1	1207	3321	0.43	100	15835
2	1692	7050	0.15	65	10454
3	1284	7782	0.18	74	7185
4	206	8610	0.90	72	1041
5	663	4381	0.40	73	6587
6a	115	1523	0.07	75	3283
6b	749	11017	0.07	75	2963
7a	541	3308	0.12	66	7121
7b	189	2656	0.12	66	3091
8a	418	3206	0.12	66	5677
8b	265	3422	0.12	66	3369
9a	352	3235	0.13	70	4743
9b	273	4023	0.13	70	2960
10a	267	3071	0.23	73	3791
10b	227	3193	0.23	73	3093
11a	72	1592	0.59	73	1960
11b	232	3339	0.59	73	3023
12	22	318	2.07	84	3045
13	28	596	1.92	80	2065
14	5	290	2.00	85	762
15	85	1651	0.65	72	2237
16	31	633	0.49	48	2135

7.2 SWMM Model Hydrology Set-up

The drainage map (sub-basins and their attributes) were imported into SWMM model. The SWMM model was then set-up with the following parameter assumptions:

- Infiltration Method = Green-Ampt
- Rounding Model = Dynamic Wave
    - Inertial Terms = Dampen
    - Normal Flow Criterion = Slope & Froude
    - Force Main Equation = Hazen-Williams
    - Min. Variable Time Step = 0.5 seconds
- Flow Routing = enabled

Furthermore, one rain gauge was inserted into the SWMM model. All sub-basins utilize the same rain gauge. This ensures that the rainfall is evenly distributed across the whole Watershed.

7.2.1 SWMM Model Validation Tests

Since there is no existing metered rainfall or flow data within the Galveston Watershed, there is no way to calibrate or formally validate the Galveston SWMM model. However, several tests against other theoretical methods were performed for investigatory purposes. The peak flow results from the Galveston SWMM model were compared to results from: (A) meter results from similar sized coastal urban area in, NJ, this did not lead to a useful comparison, perhaps due to different slopes or other regional factors; (B) results using the Rational Method; and (C) results using the Regression Estimate method for the Houston area (USGS, 1980). The testing showed that the resulting run-off rates from the Galveston SWMM model followed the same trend as the results from the three other methods, however, the magnitude of the results did not match well (this is to be expected considering the parameters/coefficients/equations used for each method are different). Results from the base 25-yr rainfall testing are compared in for the Galveston SWMM model, the Rational Method, and the Regression Estimate method (USGS, 1980). As can be seen the SWMM results fall between the other two methodologies, showing the model produces results within the range of other accepted methodologies. As expected, the rational method is more conservative and the regression equations yield lower peak runoff rates.

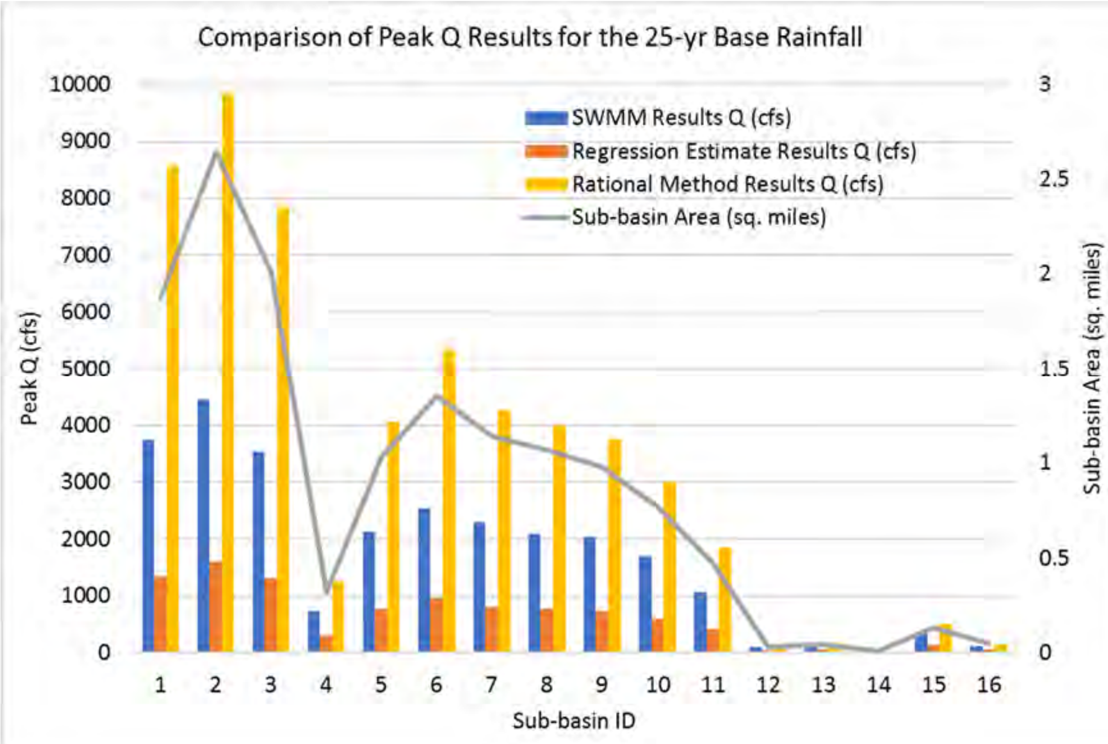

Figure 39. Peak flow results from the Galveston SWMM model and the Regression Estimate method per sub-basin using the 25-yr base rainfall; the size of each sub-basin is plotted as well for reference.

7.3 SWMM Model Hydraulics Set-up

The SWMM model was then modified by adding in hydraulic features to determine required pump sizes based on rainfall scenarios. The USACE initially provided four proposed pump locations for Galveston, all of which were located in the northwestern area of the watershed. These proposed sites are labeled A through D in Figure 40. However, during the site visit conducted in April 2018 by Mott MacDonald engineers, sites B and C were found to be challenging locations. Due to this, the Galveston SWMM model was developed in two phases – Phase 1 (preliminary) model development utilized the two pumps at locations A and D, this resulted in very large facilities including a large diameter deep tunnel which raised constructability concerns; Phase 2 (final) model development utilized two additional pumps, but in new locations at the northeastern side of the Watershed (a total of four pumps) in attempt to reduce the size of required pumps and conveyance channels. Details for the two Phases are provided in the following sections.

Figure 40. The four pump sites initially proposed by USACE for Galveston.

7.3.1 Phase 1 Model Development (Preliminary)

The Phase 1 model development utilized pumps at proposed sites A and D. A pipe was also added into pump site D, to collect runoff along the northern perimeter of the Watershed. A schematic of the SWMM model configuration is shown . Note that the schematic only provides a representation of the hydraulic system and does not show exact physical locations of the features.

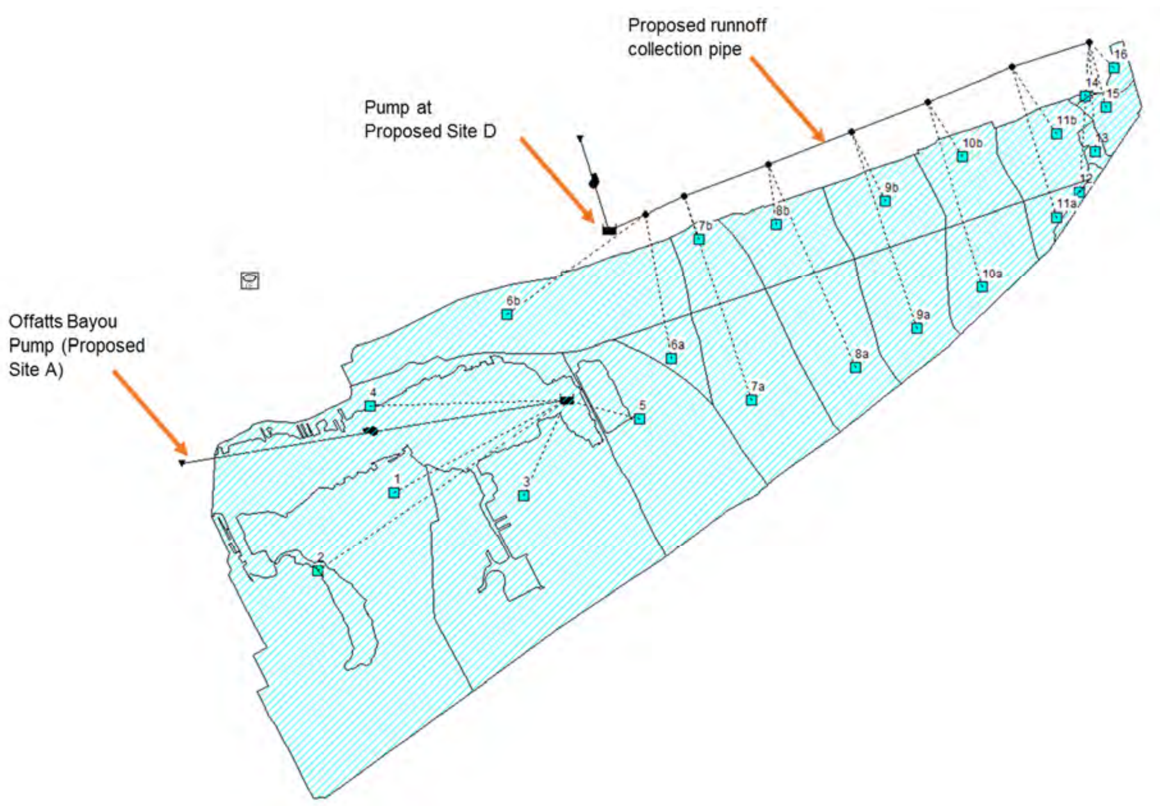

Figure 41. Schematic of preliminary Galveston SWMM model.

The runoff from sub-basin Nos. 1 – 5 was directed to the Offatts Bayou Pump. The runoff from sub-basin Nos. 5-16 was channeled to the pump at proposed site D via a collection pipe that would actually need to be a large diameter deep tunnel. The pipe serves as a wetwell for peak flow attenuation. The water body of Offatts Bayou was set as the storage unit for the Offatts Bayou pump at proposed site A.

This model was evaluated with varying rainfall conditions, which are described in more detail in Section 7.4. The following two tables provide a summary of the resulting pump and pipe configurations for each of the tested rainfall scenarios for the pump at proposed site D (Table 17) and for the Offatts Bayou pump at proposed site A (Table 18). Note that there is an unlimited number of combinations of pumping rates and pipes that could be employed. The sizes presented represent a combination that provides adequate conveyance to deliver the required flow to the pumping station while maximizing the use of the pipe for storage.

After thorough consideration of the results, these required pump and pipe sizes were determined to be infeasible due to their large sizes. Due to the large volume of run-off within the northeastern area of the watershed, it was also suspected that smaller pump sizes could be used if more pumps were implemented within this northeastern area. This triggered the Phase 2 model development, which investigated the use of two additional pumps at new locations within the northeastern sides of the watershed.

Table 17. Preliminary model analysis summary of resulting required pump and pipe sizes for the pump at proposed site D within the Galveston Watershed due to the 24-hr rainfall.

Rainfall Return Period	Required Pump Size (cfs)	Required Pipe Size (ft diameter)
10-yr + 30%	10,000	25
25-yr + 30%	10,500	30
	11,500	28
100-yr + 30%	15,000	35

Table 18. Preliminary model analysis summary of resulting required pump sizes for the Offatts Bayou pump (proposed site A) and resulting maximum water level in the Bayou within the Galveston Watershed due to the 24-hr rainfall.

Rainfall Return Period	Required Pump Size (cfs)	Max Water Elev. In Bayou (ft NAVD88)
10-yr + 30%	50	2.5
25-yr + 30%	75	3.3
100-yr + 30%	2,000	3.25

7.3.2 Phase 2 Model Development (Final)

In attempt to reduce the size of the required pump and pipe facilities, two new sites at the northeastern end of the Watershed were identified as potential additional pump sites. One of the newly proposed sites is at the lawn and parking lot adjacent to Wharf Road between 20th and 21st Streets. The other newly proposed site is at the parking lot adjacent to the helicopter landing pad and Harborside Drive, east of 11th Street. These two areas were chosen for the following reasons: pumps at the northwest end of the watershed would help relief the required size of the pump at proposed site D; they appear to provide relatively spaces adjacent to the Galveston Channel waterway; and would result in minimal disturbance to existing infrastructure and resources. Aerial images of the two sites are shown in Figure 42. Mott MacDonald has coordinated with and obtained permission from the USACE to recommend these two additional sites as proposed pump site locations. Further investigation into real estate requirements should be conducted in future phases of this study.

Figure 42. Aerial of the Phase 1 pump locations (blue rectangles) and the two new proposed pump sites (highlighted with yellow rectangles) at the northeastern side of the watershed; the Phase 2 analysis uses all four pump sites shown in this aerial.

Thus, the Galveston SWMM model was modified to include pumps at the two new sites, in addition to the pumps at sites A and D. Rectangular channels were also incorporated into the model to collect run-off from the sub-basins and divert it into the desired pump station. Due to the large storage capacity offered by Offatts Bayou, runoff from more sub-basins is diverted into the Bayou by using a channel along Broadway Ave. Locations of the conveyance channels were selected based on wide streets that appeared to provide the required space. As no utility information was available utility conflicts and relocations will need to be addressed in future project phase. Each channel also serves as a wetwell during a rainfall event. A schematic of the modified Galveston SWMM model with the new pump configuration, and with a new pump site naming convention, is shown in Figure 43. After testing the model with the range of rainfall scenarios, it was found that this layout successfully reduced the pump sizes from the Phase 1 (preliminary) model results, and allow for shallow conveyance conduits that could be constructed using cut and cover practices. Therefore, all results for proposed conditions for Galveston provided within this report henceforth, were calculated from this modified (final) version of the model.

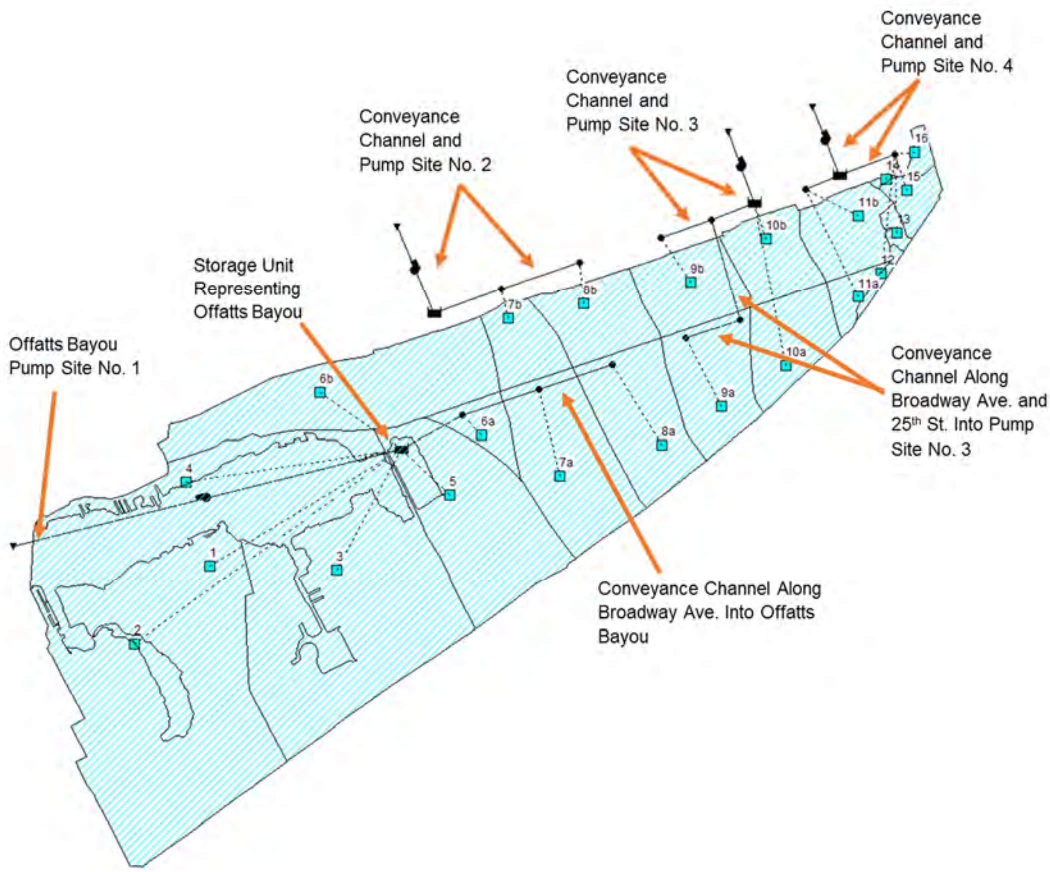

Figure 43. Schematic of the modified (final) Galveston Watershed SWMM model. Note that the model only serves as a visual representation of the system and does not show actual physical locations of the proposed channels or pumps.

7.4 Evaluation Scenarios

The Galveston Watershed SWMM model was run for each of the scenarios discussed in Section 4. For the model scenarios that tested the proposed 125' navigation gate (at the mouth of the Offatts Bayou) in the closed position, the Offatts Bayou storage unit water level was lowered to -1 ft NAVD88, which is approximately 1 ft below mean low water (MLW) elevation, at the start of the simulations. This simulates the situation of draining the Bayou before the storm makes impact, which allows it to hold more runoff during the duration of the storm. This in turn allows a smaller pump size to be used for the Offatts Bayou pump. It's important to note that change in water levels due to SLR will impact the storage capacity of the Bayou. Similarly, an increase in water level due to SLR will require the pump to be turned on earlier prior to the storm, in order to lower the water level inside the Bayou down to -1 ft NAVD88. For the model runs that tested the proposed gate in the open position, the initial water level was set to 0.87 ft NAVD88, which is equivalent to the local mean high water (MHW) elevation. It should be noted that as sea levels rise it will take longer and longer to dewater the Bayou in advance of a storm. Additional pumping facilities may be required to dewater the bayou in a timely manner and the proposed pumping station should include space for expansion or be oversized accordingly.

For each simulation, the pump sizes and channel dimensions were optimized based on the following limiting conditions:

- the water level in the channels and storage units could not rise above ground elevation (channels were typically modeled with a 5 ft ground clearance); and
- the water level in the Offatts Bayou could not rise above elevation +4 ft NAVD88 (which is the top of bank elevation around the Offatts Bayou shoreline according to site topographic data (see Figure 44); this minimizes flooding into the properties adjacent to the Bayou).

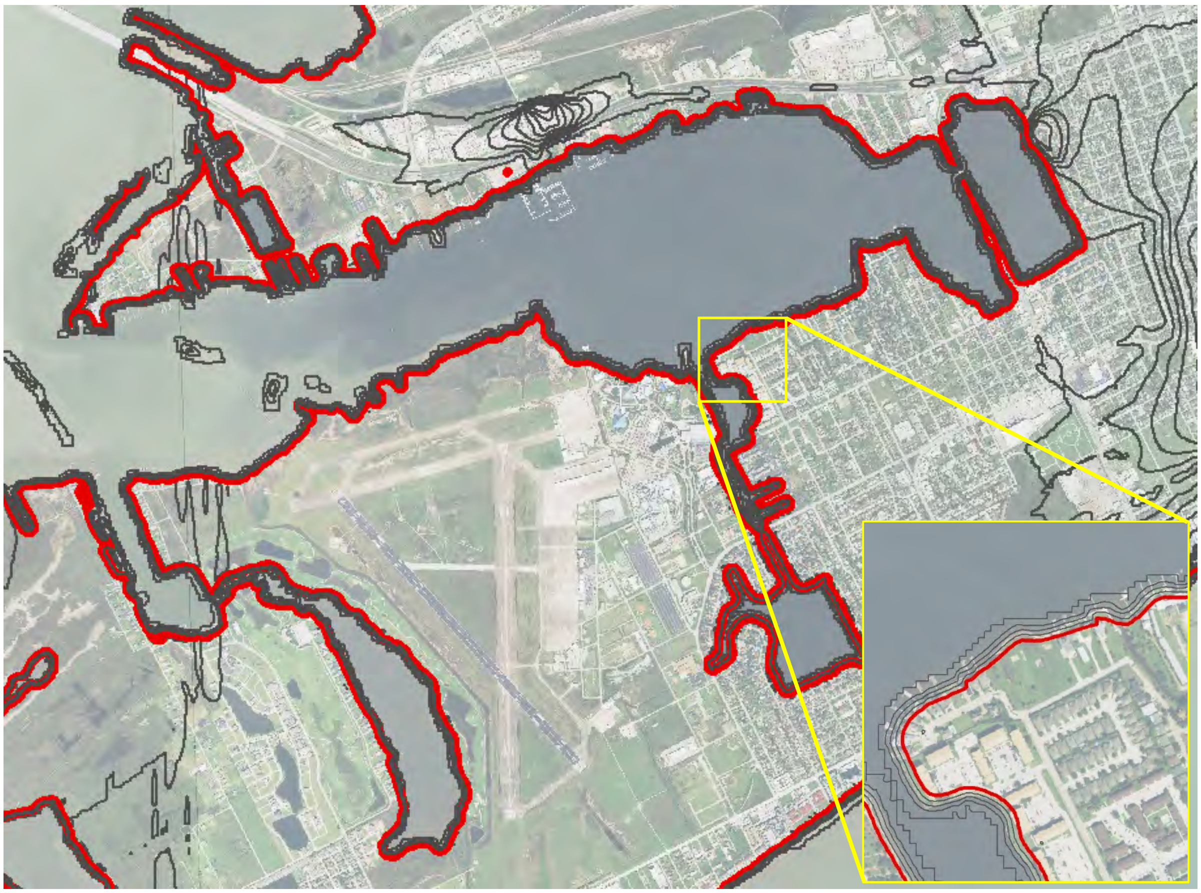

Figure 44. Contour map of Offatts Bayou, with the +4 ft NAVD88 contour line shown in bold red, other contour lines in black. An insert in the lower right corner shows a zoomed in view of the +4 ft NAVD88 contour line at the top of the shoreline bank.

7.5 Analysis & Results

The following sections provide description and results for model runs discussed in Section 4.

7.5.1 Existing Conditions

Since there was no available information on the existing interior drainage system within Galveston, assumptions must be made as to how and where the run-off within the watershed flows for the existing conditions. Based on the contours within the watershed (see Figure 45). it is assumed that run-off from sub-basins surrounding Offatts Bayou flows directly into the Bayou. Run-off from all other sub-basins is assumed to flow north into the adjacent Galveston Channel Waterway (because the flood barrier is not present in existing conditions). This is a valid assumption as it is unlikely the existing storm drainage system has a 25-year capacity and likely the majority of the flow will be overland flow, following the surface contours. Due to this, comparisons between existing and proposed conditions can only be evaluated for the resulting water elevation inside Offatts Bayou from the run-off draining into it. Therefore, the resulting peak water levels inside Offatts Bayou were investigated under existing conditions for the following rainfall events: 10-yr+30%, 25-yr+30%, 50-yr+30%, 100-yr+30%, and 500-yr+30%. To model the existing conditions, a cross section of the mouth of the Bayou was developed using depths from the NOAA Navigation Chart No. 11324. A water elevation of 0.87 ft NAVD88 was used as a boundary condition at the mouth of the Bayou. This simulates the bay at mean high water (MHW) during the rainfall event. The resulting peak water surface elevations in the Bayou is provided in Table 19. The results indicate that during existing conditions, the water elevation does not rise much under the varying rainfall events, because the water is able to flow out of the Offatts Bayou mouth and into Galveston Bay.

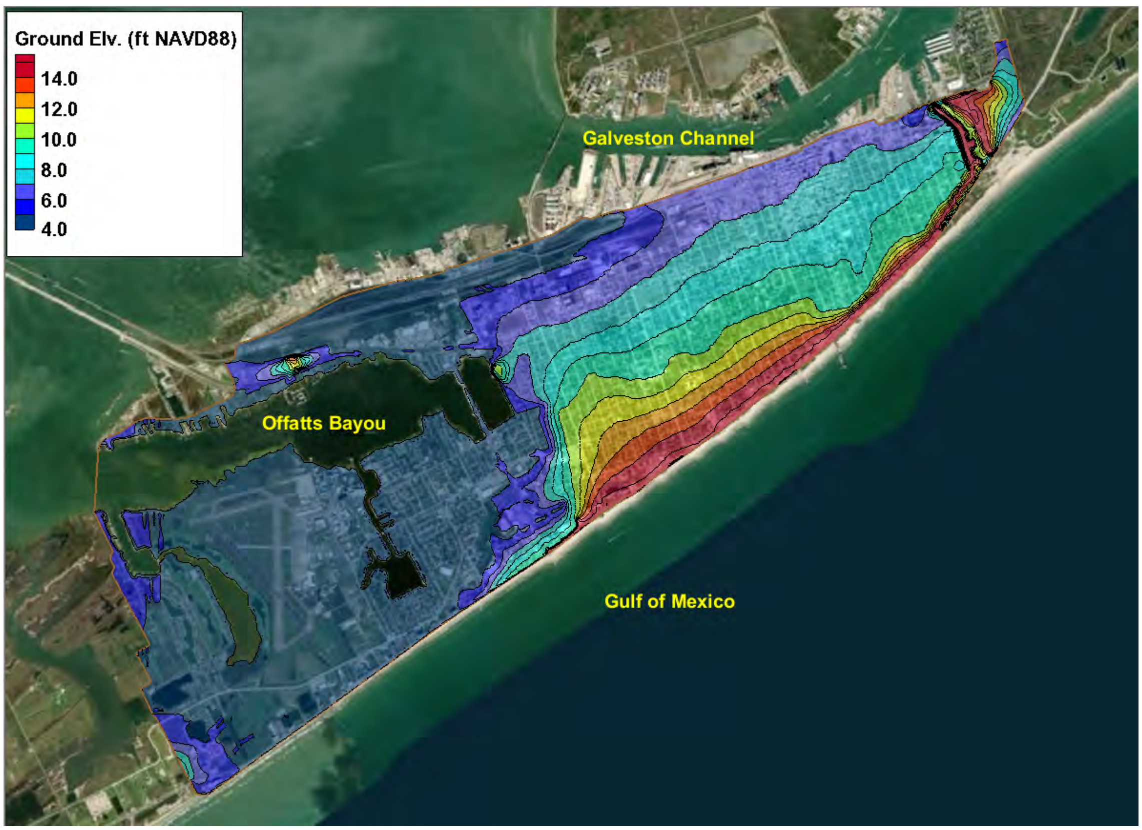

Figure 45. Contours showing the ground elevation within the Watershed; contours are based on elevation data from USGS (2018).

Table 19. Resulting peak water elevations within Offatts Bayou under existing conditions. Note that these runs only evaluated the run-off from sub-basins that are assumed to flow directly into the Bayou.

Rainfall Event	Offatts Bayou Peak Water Elevation (ft NAVD88)
10-yr+30%	0.87

Rainfall Event	Offatts Bayou Peak Water Elevation (ft NAVD88)
25-yr+30%	0.87
50-yr+30%	0.87
100-yr+30%	0.87
500yr+30%	0.88

7.5.2 Facilities Development for Design 25-yr+30% Rainfall Event

The proposed pump and conveyance channels for the Galveston SWMM model were then optimized for the 25-yr+30% rainfall scenario. The channels draining into pump site nos. 2 – 4 were modeled with approximately 5 ft of ground coverage and a zero percent slope. The channel draining into Offatts Bayou was modeled with a 0.1 % slope and varying ground clearance. The invert of the channel at the intersection of Offatts Bayou was set to -16 ft NAVD88. This elevation was developed based on available bottom elevations within the Bayou found in NOAA Navigational Chart No. 11324. Table 20 provides a summary of the resulting required pump and channel sizes. With these facility sizes in place, the peak water elevation in Offatts Bayou for the 25-yr+30% rainfall event is 2.5 ft NAVD88. Since the 25-yr+30% rainfall event is considered the design rainfall for this scope of the project, these resulting pump and channel sizes are considered the design facilities for the City of Galveston.

Table 20. Resulting pump and channel sizes for the 25-yr+30% rainfall scenario. The resulting peak water elevation in Offatts Bayou with these facilities in place is 2.5 ft NAVD88.

Pump Site No.	Pump Size (cfs)	Channel Width (ft)	Channel Height (ft)
1 (Offatts Bayou)	250	30	13
2	1500	20	10
3	4500	20	10
4	1500	20	10

7.5.3 Facility Sizes for Other Rainfalls

The Galveston SWMM model was further tested with the remaining rainfall scenarios to determined required pump and channel sizes based on rainfall event. Each of these simulations was run with the proposed gate closed, pumps on, and initial water level in Offatts Bayou set to 1 ft below MLW. Graphs were developed for each pump site (nos. 1 – 4) and their associated conveyance channels to summarize the results. Figure 46 shows the required facility sizes per rainfall event for the pump and conveyance channel at the Offatts Bayou pump site no. 1, and Figure 47 shows the resulting max water elevation in Offatts Bayou based on these facility sizes. While Figure 48 though Figure 50 provide the resulting facility sizes per rainfall event for pump site nos. 2 – 4, respectively.

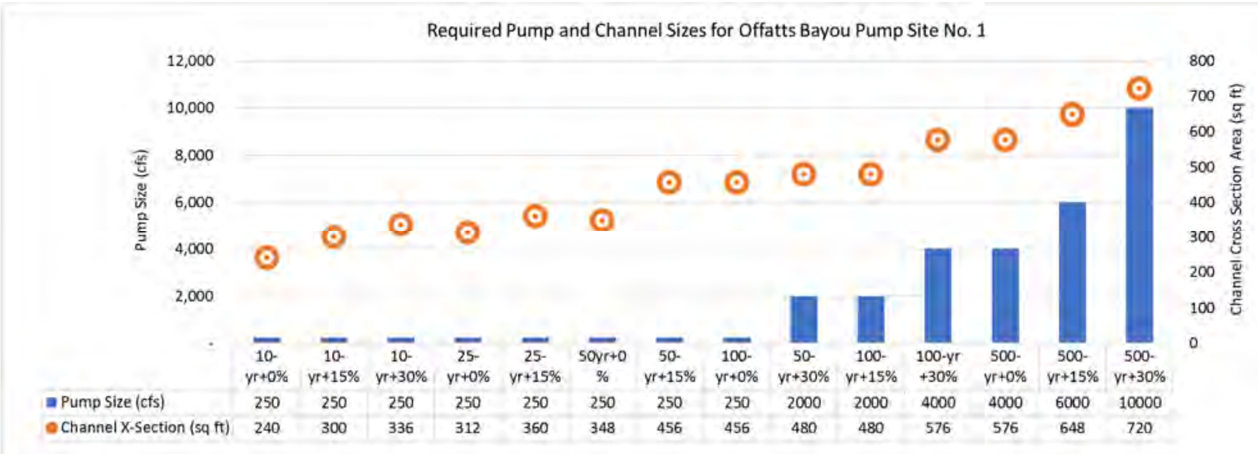

Figure 46. Required pump and channel sizes for the Offatts Bayou Pump Site No. 1.

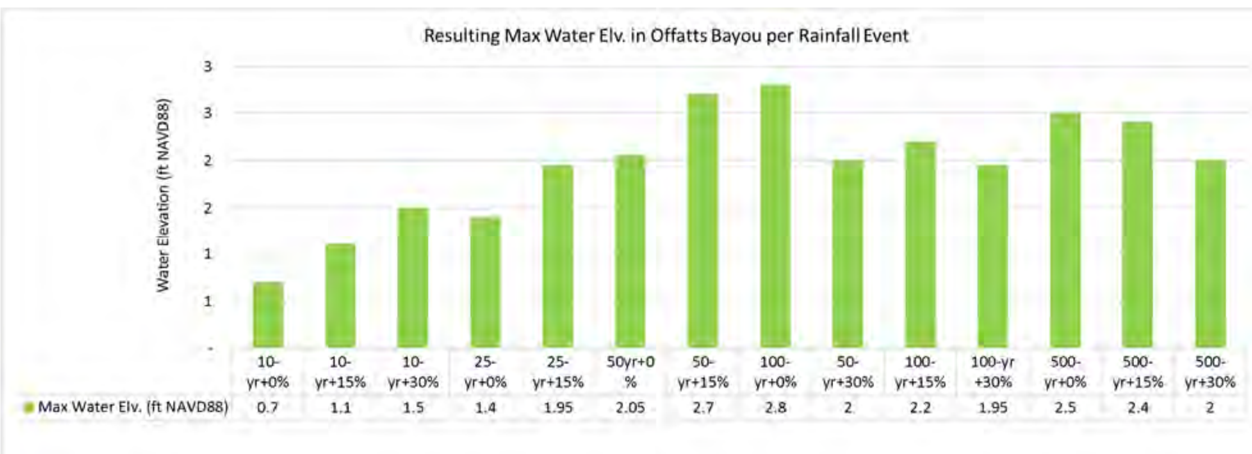

Figure 47. Resulting max water elevation in Offatts Bayou based on the pump and channel sizes shown in Figure 46 per rainfall event.

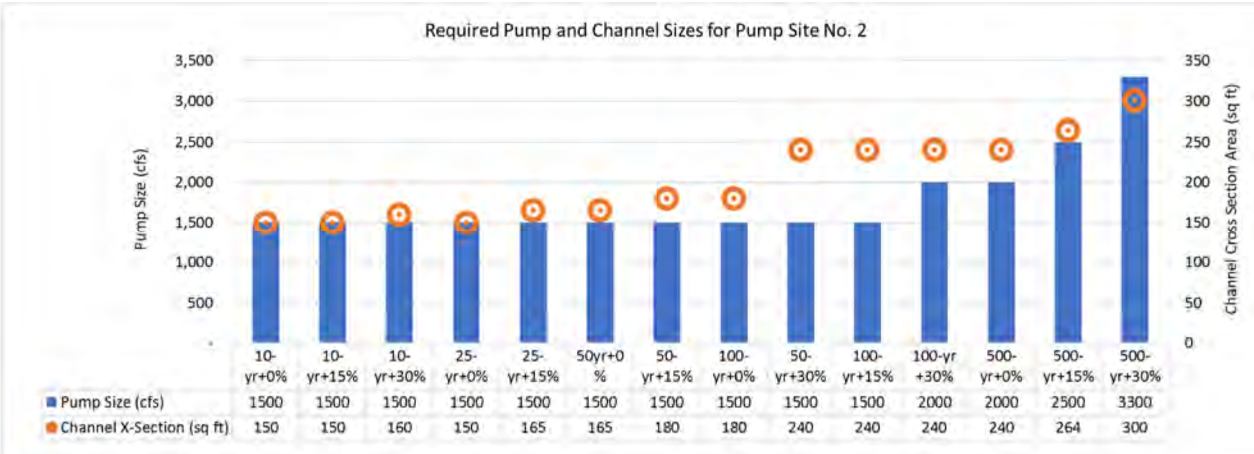

Figure 48. Required pump and channel sizes for Pump Site No. 2.

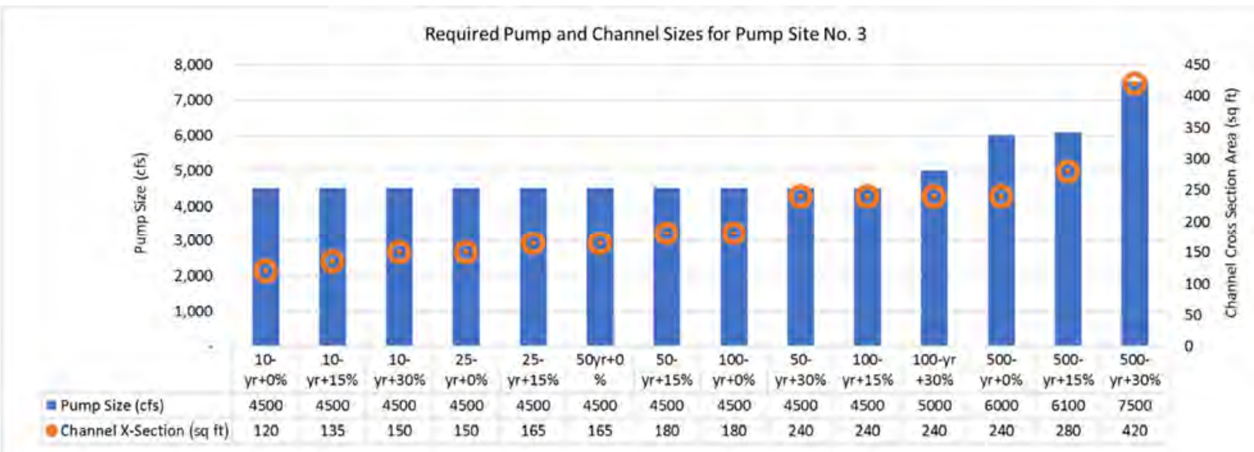

Figure 49. Required pump and channel sizes for Pump Site No. 3.

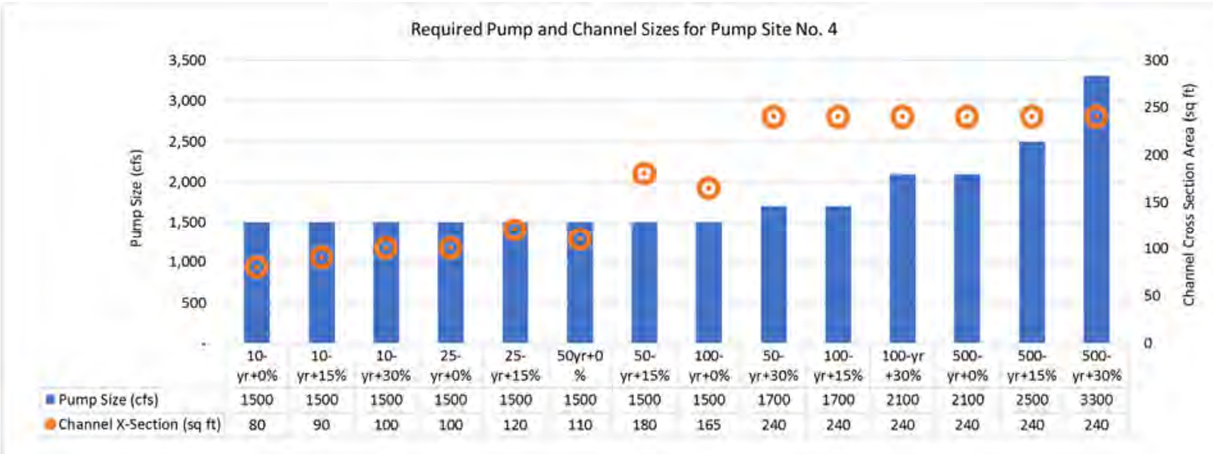

Figure 50. Required pump and channel sizes for Pump Site No. 4.

7.5.4 Comparison of Existing to Proposed Conditions

In order to compare performance of the Galveston Watershed under existing conditions to that of proposed conditions, only the peak water level in Offatts Bayou can be evaluated (see Section 7.5.1) The four scenarios, in which the resulting peak water elevations in Offatts Bayou are compared, are as follows:

- Base – existing conditions
  - With sub-basins surrounding Bayou draining directly into it
  - No channels modeled
  - With MHW boundary condition (MHW = 0.87 ft NAVD88)
- Case 1 – Proposed: gate closed, pumps on
  - With proposed flood barrier
  - With 25-yr+30% design pumps and channels
  - With proposed 125 ft gate closed
  - With initial water level in Bayou 1 ft below MLW
- Case 2 – Proposed: gate open, pumps on
  - With proposed flood barrier
  - With 25-yr+30% design pumps and channels
  - With proposed 125 ft gate open
  - With MHW boundary condition
- Case 3 – Proposed: gate open, pumps off
  - With proposed flood barrier
  - With 25-yr+30% design channels
  - With proposed 125 ft gate open
  - With MHW boundary condition

The following Figure 51 provides a graph summarizing the results for each rainfall event. Each rainfall event with the four cases is shown on the x-axis and the resulting peak water elevation in Offatts Bayou is on the y-axis. Also, recall that the highest contour at the shoreline bank of Offatts Bayou is 4 ft NAVD88; therefore, elevations higher than 4 ft NAVD88 indicate that the surrounding land will be flooded.

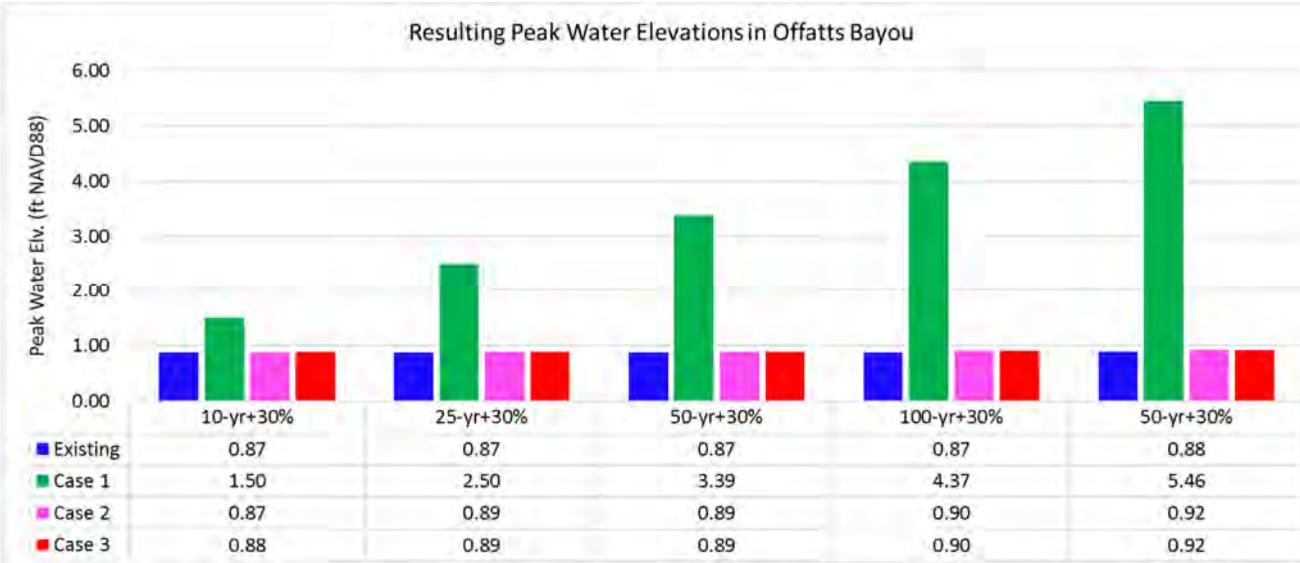

Figure 51. Comparison of resulting peak water elevations in Offatts Bayou for each scenario per rainfall event; these simulations only evaluated run-off from select sub-basins within the watershed.

As evident from the plot, the proposed Cases 2 and 3 result in little change from the existing conditions (Base Case) for each rainfall event. The resulting Offatts Bayou max water elevations are observed to slightly increase because more run-off is directed into the Bayou in the proposed conditions via the proposed conveyance channel along Broadway Ave. Furthermore, the proposed Case 1 (with the gate closed, pumps on), does cause a greater increase in max water elevation at the Bayou when compared to the existing conditions. However, because the highest contour along the bank of the Bayou's shoreline is 4 ft NAVD88, the rise up to 1.5 ft NAVD88 and 2.5 ft NAVD88 for the 10-yr+30% and 25-yr+30% rainfall, respectively, is considered acceptable. As discussed previously it is unlikely the navigation gates will be closed for precipitation events greater than the 25-year storm.

It is important to note that flooding from all sub-basins not draining into the Bayou is not considered in these companions. With the flood barrier in place for the proposed conditions, it's likely that flooding from the remaining sub-basins will flow over land and make its way into Offatts Bayou. However, simulating this surface flow is not included within this scope, and it is assumed that this will be evaluated in more detail at later stages of the project.

7.5.5 Flood Elevation Impact: Gate Closed, Pumps On

The Galveston SWMM model was then tested with the design 25-yr+30% facilities (pumps and channels) in place, against the 10-yr+30%, 50-yr+30%, 100-yr+30%, and 500-yr+30% rainfall, to evaluate potential flooding impacts within the proposed floodwall (with the navigation gate closed). The EPA SWMM program does not produce total stillwater flood elevations across a watershed; however, it does provide flood volumes at each node within the modeled system. Therefore, to evaluate this, a surface was developed using Surface-water Modeling System (SMS) with the topography dataset from USGS (2018). The maximum flood volumes from each node in the SWMM simulations were summed to find the total flooded volume during each 24-hr rainfall scenario. Then, using the surface, the entire Galveston Watershed (within the proposed floodwall alignment) was treated as a single storage unit. The storage unit was filled with the total flood volume quantities from the SWMM simulations, and the resulting flood elevations were identified. Results for each run are as follows:

A. 10-yr+30% rainfall with 25-yr+30% facilities: No flooding expected to occur.
B. 50-yr+30% rainfall with 25-yr+30% facilities: Temporary flooding expected to occur within the watershed, but due to the topography, flood volumes are expected to flow over land and drain into Offatts Bayou.
C. 100-yr+30% rainfall with 25-yr+30% facilities: Flooding expected to occur; flood stillwater elevation expected to reach approximately 4.4 ft NAVD88.
D. 500-yr+30% rainfall with 25-yr+30% facilities: Flooding expected to occur; flood stillwater elevation expected to reach approximately 5.5 ft NAVD88.

The following figures show the approximate standing flood stillwater coverage for the 100-yr+30% rainfall (Figure 52) and the 500-yr+30% rainfall (Figure 53). As discussed previously it is unlikely the navigation gates will be closed for precipitation events greater than the 25-year storm, mitigating this risk. Note that these images represent the area of land coverage that is below the 4.4 ft and 5.5 ft NAVD88 elevations, respectively. Therefore, this coverage represents the standing floodwater, after run-off from throughout the watershed flows over land and into the Bayou. It's possible for surface water to reach higher elevations while it flows over land. Furthermore, ponding floodwater may exist elsewhere across the watershed depending on local topography or surface/structural features. However, in order to investigate to that level of detail, a more recent and higher resolved topography or DEM dataset would be needed. It's expected that an analysis to that level of detail will be performed during the next phase of the project, since it is not included in this scope of the project.

Figure 52. Standing flood coverage at elevation 4.4 ft NAVD88 from the 100-yr+30% rainfall with the 25-yr+30% facilities.

Figure 53. Standing flood coverage at elevation 5.5 ft NAVD88 from the 500-yr+30% rainfall with the 25-yr+30% facilities.

7.5.6 Tidal Range Test

To test the impact of the proposed floodwall and 125 ft long navigation gate at the mouth of the Offatts Bayou, the model was run with existing conditions (without the presence of the floodwall and gate) and with the proposed floodwall and gate. To perform this simulation, the model was run with a 3-day long tidal period from the local NOAA tide gauge (see Section 4 for more details). Figure 54 provides a plot of the resulting water levels inside and outside the Bayou for the existing (no gate) and proposed (with gate open) conditions. A comparison of results indicates that the presence of the floodwall with the gate open has little effect on the tide range inside Offatts Bayou.

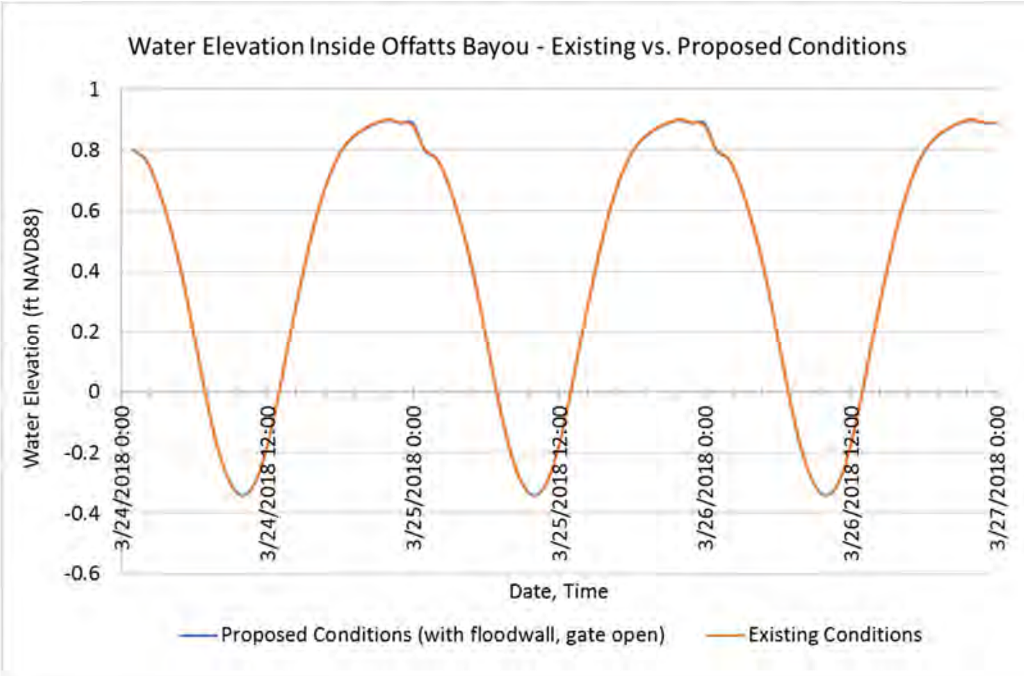

Figure 54. Resulting water levels inside Offatts Bayou over a 3-day tidal period with existing conditions (orange line) versus proposed conditions (with the floodwall and navigation gate open) (blue line).

7.5.7 Overtopping Analysis

An overtopping analysis was conducted along the Galveston Ring Levee. This analysis was conducted to determine whether overtopping of the proposed seawall improvements and ring levee causes any additional flooding on the Island. Overtopping of coastal structures is highly dependent on both the cross-sectional design of the protection element and the ocean conditions during a storm event. Currently, final design of the Galveston Island Ring Levee and Galveston Seawall improvements has not been conducted by the USACE. The USACE has provided preliminary cross sections for each feature, which are shown below in Figure 55. These preliminary cross sections were used to calculate overtopping rates. Any changes to the cross-sectional design shown in Figure 55 will result in changes to the overtopping rates provided in this Section.

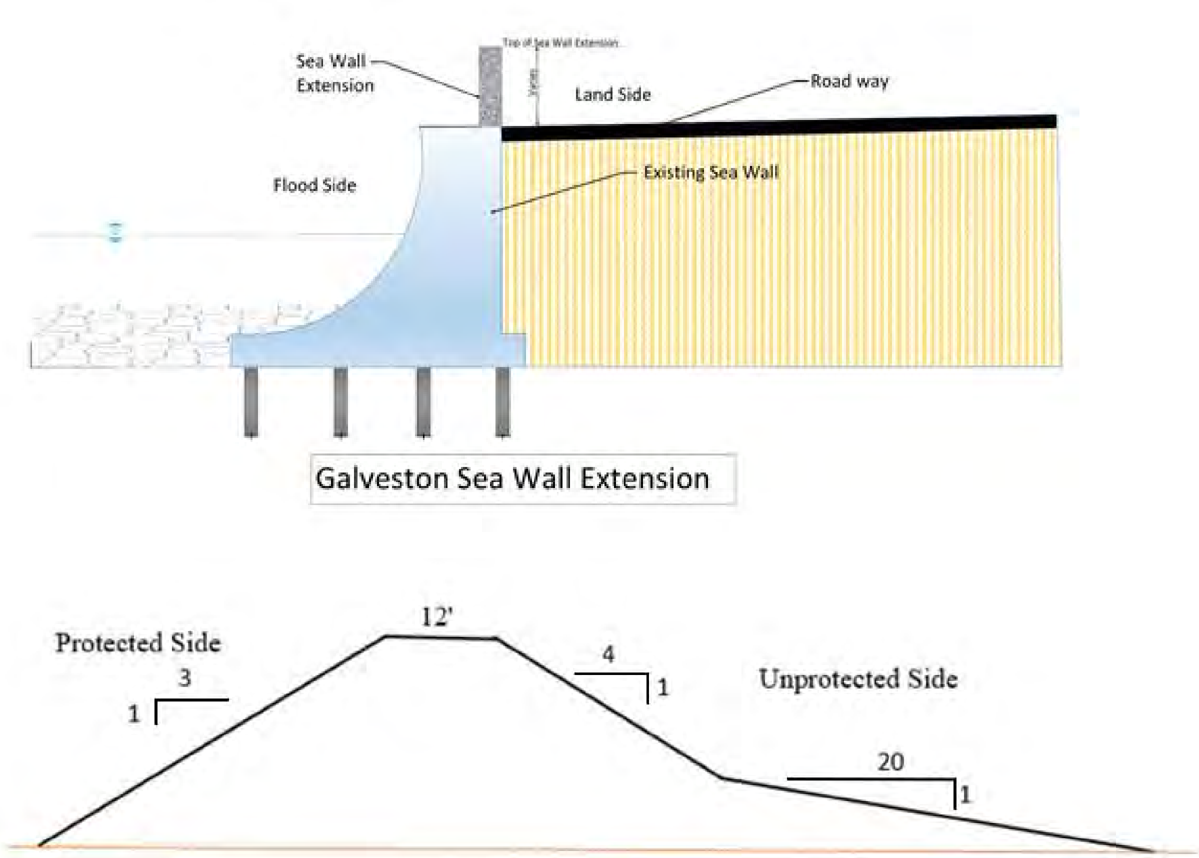

Figure 55. Preliminary cross sections provided by the USACE for the Galveston Seawall Extension (top) and Galveston Ring Levee (bottom).

Wave overtopping is also highly dependent on the top elevation of the protection structure. Currently, the USACE has not optimized the levee or seawall extension heights. To allow Mott MacDonald to perform preliminary overtopping calculations, the USACE provided the levee heights used in the ADCIRC model. The levee and seawall improvement heights provided by the USACE are shown below in Figure 56.

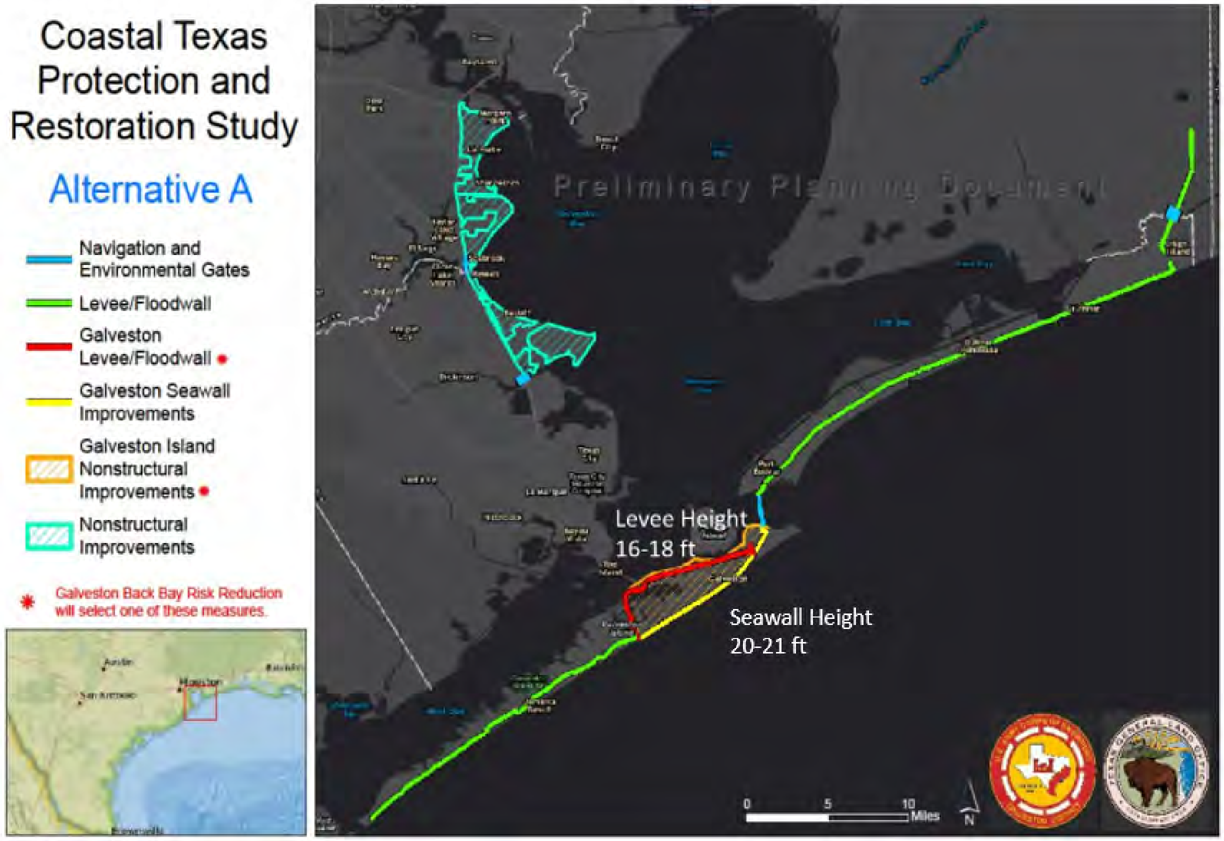

Figure 56. Levee height depiction provided by the USACE. Note that the

As shown in Figure 56, the top of seawall elevation provided was 20-21 feet. Mott MacDonald assumed a seawall improvement top elevation of +21 feet NAVD88 for the overtopping analysis. The top elevation of the Galveston Ring Levee on the backside of the Island was preliminary set to 16-18 feet. For this analysis, the top elevation of the levee was assumed to be +17 feet NAVD88.

The USACE provided timeseries outputs of waves and water surface elevations for the 20 storms comprising the Joint Probability Method suite. These storms were simulated using a coupled ADCIRC-STWAVE model. Four representative points for overtopping calculations were selected by Mott MacDonald from the ADCIRC-STWAVE model output provided by the USACE. Overtopping rates were calculated at each of these four representative points. The overtopping rates calculated at each point were then applied to the representative shorelines shown in Figure 57. The representative shorelines were chosen to depict areas where similar wave conditions could be expected. Figure 57 shows the representative points as well as the representative shorelines along which the overtopping rates were applied.

Note the southern portion of the representative shoreline for Point 1, as well as the eastern shoreline for representative Point 4 likely experience less energetic wave regimes than those at the extraction point. At this stage of preliminary design, the overtopping rates calculated at the extraction point were applied to these sections to calculate a conservative estimate of overtopping volumes.

Figure 57. Model extraction points and representative shorelines where overtopping rates were applied.

The USACE modeled 20 storms comprising the JPM-OS storm suite using a coupled ADCIRC-STWAVE model. To determine the 100-year overtopping rate, the peak water surface elevation during each storm timeseries was compared to the 100-year, 90%, 2085 water surface elevation. A summary of the 100-year, 90% Confidence Interval, 2085 water surface elevations at each of the four extraction points is shown below in Table 17.

Table 21: Extraction Point extremal WSEs. 100-year, 90% CI, 2085 WSE shown

Point	WSE [ft NAVD88]
1	9.1
2	7.5
3	11.8
4	17.3

The storm with the WSE closest to the 100-year event was selected as the design overtopping event. The timeseries of waves heights, water surface elevations, and peak periods associated with this event were then extracted from model results. To determine the wave conditions at the structure, the waves at each extraction point where then shoaled to the base of the structure for input into the overtopping equations. Input conditions for all extraction points are shown in Appendix A.

Equations from the Eurotop, 2016 manual were used to calculate an overtopping timeseries along the proposed seawall and levee improvements for the design storm. A neural network tool developed by Eurotop 2016 was also tested at each extraction point (Eurotop, 2016, Formentin et al., 2017, Zanuttigh et al. 2016). Along the levee side of the island, neural network results showed little correlation with the analytical results and very large confidence bounds. This indicates a poor fit of the neural network tool, and therefore the analytical results were used to calculate overtopping along the levee portion of the island. Equations 5.12 and 5.13 of

Eurotop, 2016 were used to calculate overtopping rates along the Galveston Bay Side of the Ring Levee.

The structural parameters used as inputs into Equations 5.12 and 5.13 are shown below in Table 22. Appendix A shows the full set up inputs, including depth, wave height, wave period and WSE, used to calculate overtopping at all extraction points.

Table 22. Input Parameters used in Overtopping calculations at points 1, 2, 3.

Parameter	Value
Top of Structure [ft NAVD88]	17 ft
Slope	4H:1V
Crest Width [ft]	12 ft
γ_f	0.6
γ_{beta}	1.0
γ_b	1.0
γ_v	1.0

The top of structure, slope and crest width shown in Table 23 were taken from the USACE conceptual cross section shown in Figure 55. The other parameters are reduction factors that account for surface roughness (γ_f), wave obliqueness (γ_{beta}), berm width (γ_b), and wavewall construction (γ_v). A γ_f value of 0.6 was selected, which corresponds to one layer of armor stone on the unprotected side of the levee, with an impermeable core. If the design of the unprotected side is altered to include a different covering type, this value should be updated. All other reduction factors were set to 1.0 for conservatism. The peak overtopping rates for the design event at point 1, 2, 3 are shown below in Table 23. These overtopping rates are negligible compared to the rainfall during the design event and are not expected to impact the required pump capacity. Any changes to the cross-sectional design of the Galveston Ring Levee will require a re-analysis of overtopping effects on pump capacity.

Table 23. Peak overtopping rates for 100 year (2085 90% Confidence Interval) design event along the Bay-Side of proposed Galveston Ring Levee.

Point	Peak Overtopping Rate [cfs/ft]
Point 1	$8.2E10^{-7}$
Point 2	0
Point 3	$2.7E10^{-7}$

Along the proposed seawall improvement, the neural network tool showed similar overtopping rates when compared to the analytical equations (Equation 5.18 of Eurotop, 2016). The neural network tool showed slightly more conservative (i.e. higher) overtopping rates and was therefore used to compute overtopping volume along the proposed seawall expansion. See Figure 58 for a comparison between the analytical and neural network results.

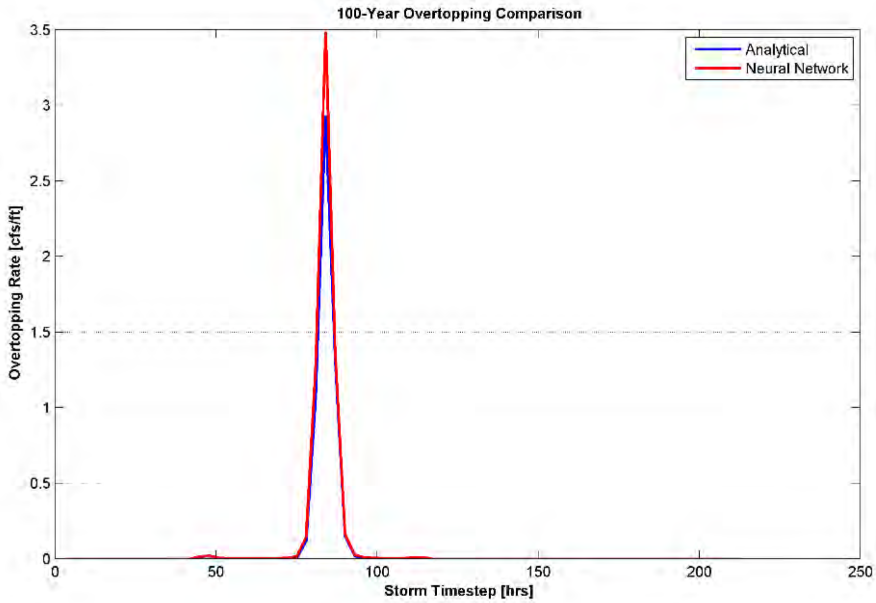

Figure 58. Comparison between overtopping timeseries for neural network (red), and analytical equations from Eurotop, 2016 (blue).

The seawall expansion proposed by the USACE includes adding a 4-foot vertical wall to the existing seawall design. This increases the top elevation of the seawall from approximately +17 ft to +21 ft NAVD88. The peak overtopping rate experienced during the design event for the current design are greater than 3.4 cfs/ft. This large overtopping rate has the potential to cause significant damage to infrastructure behind the seawall and to drastically increase the pumping requirements of the Galveston Pump Stations. Therefore, Mott MacDonald investigated combinations of raising the seawall and adding a return wall to reduce overtopping. Through sensitivity testing, Mott MacDonald found that a 1-ft-high by 3-foot-wide return wall yielded the greatest reduction in overtopping volume. The results of the testing are shown below in Table 24.

Table 24: Overtopping flowrates and volumes for varying top elevations of seawall improvement. Testing conducted with and without 1 ft high by 3 ft wide return wall.

Scenario	T.O. Seawall Elevation [ft NAVD88]	Peak Overtopping Flowrate [cfs/s/ft]	Peak Overtopping Volume* [cfs/s]
No Return Wall	21	3.49	147,919
	22	2.50	105,917
	23	1.74	73,960
	24	1.18	50,219
	25	0.78	33,236
With Return Wall	21	1.85	78,591
	22	1.88	79,768
	23	0.35	14,705
	24	0.23	9,911
	25	0.15	6,207

*Volume assumes a total seawall length of ~42,000 lf.

Based on the results shown in Table 24, it is recommended that the final design conducted by the USACE investigate raising the seawall higher than +21 ft NAVD88. In addition, the return wall shows large reductions in overtopping flowrate, and is recommended for further investigation.

Mott MacDonald used the EPA SWMM model described in Section 7.2 & 7.3 to calculate the required pump size if overtopping was included in the pumping requirement calculations. Mott MacDonald assumed that all overtopping flow would be collected along the seawall and routed directly into Offatts Bayou via a separate pipe or channel. The proposed Offatts Bayou pump and Broadway Street conveyance channel were then sized to accommodate each overtopping value. Note that this is a preliminary analysis for discussion only. Due to the uncertainty regarding the final design elevation of the seawall improvements, the final pump station design submitted by Mott MacDonald assumes that the seawall expansion would be optimized to allow no overtopping for the design event.

Table 25: Required Pump size for selected seawall improvement designs

Parameter	No Overtopping	With 100-year Overtopping Flowrates					
Top of Seawall Elevation [ft NAVD88]	21’	25’ Return Wall	24’ Return Wall	23’ Return Wall	22’ Return Wall	21’ Return Wall	21’ w/out Return Wall
Peak Overtopping Flowrate [cfs]	0	6,207	9,911	14,705	79,768	78,591	147,919
Offatts Bayou Pump Size [cfs]	250	250	2,000	4,000	8,000	10,000	60,000
Required Broadway Ave Conveyance Channel Size	30’ wide x 13’ high	30’ wide x 13’ high	30’ wide x 13’ high	31’ wide x 13’ high	34’ wide x 13’ high	34’ wide x 13’ high	35’ wide x 13’ high
Resulting Max WSE in Offatts Bayou	2.5 ft NAVD88	2.75 ft NAVD88	1.7 ft NAVD88	1.3 ft NAVD88	3.0 ft NAVD88	2.9 ft NAVD88	3.2 ft NAVD88

Note that with seawall expansions below 22’, a significant increase in the pump capacity of Offatts bayou is required. Therefore, it is recommended that the USACE investigate raising the seawall to at least +23’ NAVD88 and adding a return wall to the design. Again, note that all calculations shown here are highly variable depending on the final design of the levee and seawall expansion. Finally, note that the pump station designed by Mott MacDonald assumes that no overtopping will be allowed by the final design of the levee and seawall expansion. If any overtopping of the levee or seawall is allowed it will require re-analysis of the pump capacities and conveyance channel sizes.

7.6 Concept Plans

Aerial views and cross section plans of the conceptual pump and channel layout are provided in Appendix D and E, respectively. Typically, when local drainage pipes pass through the line of protection, they are equipped with redundant backflow prevention consisting of a tide gates and a sluice gate so that positive flow cut off can be achieved. In the case of Galveston, a mechanism should also be provided to divert flow from the existing drainage pipes into the conveyance channel to allow pumping of stormwater when the tailwater is elevated, without causing surface flooding. Figure 59 shows a schematic of a proposed connection between the existing drainage network and the proposed channel/conduit. Since the existing storm sewer are likely not designed to convey the 25-year storm, during the design event there will be a good deal of surface flow. This flow must be collected and brought into the conveyance channels. Surface collection within the Watershed may be achieved by adding inlets within close proximity to each other along the roadways to increase flow into the drainage system. An example of this is shown in Figure 60, which shows an array of inlet grates installed as part of the USACE Green Brook Flood Control Project in Bound Brook, NJ with numerous inlets to capture interior drainage during the 150-year design storm.

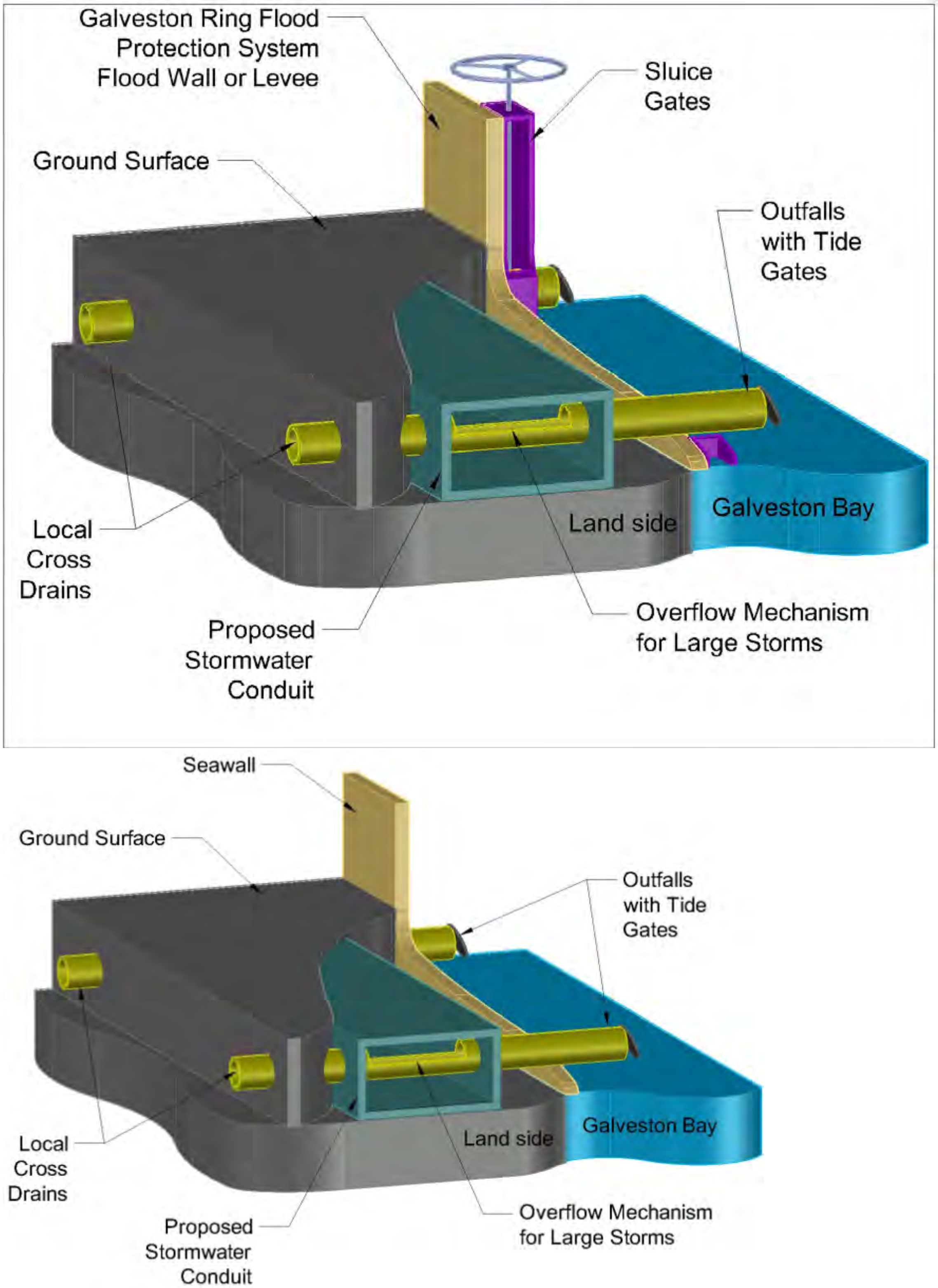

Figure 59. Proposed stormwater conduit connection with existing drainage network.

Figure 60. Google Maps Street View image of an example location with a set of inlets to capture extreme events along the road.

8 Suggested Future Analysis

8.1 General

The drainage analysis conducted in this study is highly dependent on historical rainfall and surge data. The analysis outlined in Section 3.3 was used to select the design rainfall and surge events based on historical data. The analysis for each watershed assumes that the peak rainfall and surge events coincide, and that the gate structure must remain closed the full duration of the storm event. To further refine and potentially reduce pump sizes, a Joint Probability Analysis (JPA) should be conducted correlating rainfall and surge events. It is anticipated that this process would be similar to the standard JPM-OS analysis that is currently conducted to determine extremal storm surges. Conducting this analysis could refine the design pump sizes and potentially reduce project costs.

8.2 Clear Creek

To improve the accuracy of the Clear Creek model, the existing model, which may be outdated, should be re-calibrated, to Hurrican Harvey and other extreme events, based on measured stage and flow rate at the Friendswood USGS gauge (USGS 08077600). This gage has recorded data during the 2017 Hurricane Harvey event, and calibration of the model to this particularly large event would increase the model accuracy for large return periods (100- and 500-year rainfall).

The model can be further improved by investigating the source if the stage-flow rating curve at the Friendswood USGS gauge, including the influence of a potentially looped rating curve on estimated flow rates.

8.3 Dickinson Bayou

To improve the interior drainage analysis for Dickinson Bayou, new data is required. The following recommendations are suggested to help improve the future analysis:

- A detailed field campaign for refining and confirming flow patterns and sub-watersheds boundaries is recommended for the next level of effort for Dickinson Bayou hydrology analysis.
- It is recommended to install streamflow gages at HWY 146 and at Cemetery Rd. HWY 146 represents the total drainage area of Dickinson Bayou. Cemetery Rd represents three criteria: (1) the point of largest different between Mott MacDonald and FEMA results, (2) the location on Dickinson Bayou where the general percent land urbanization changes from rural to developed, (3) the upstream limit of the HEC-RAS model (see Section 6.3). The newly acquired streamflow data in combination with precipitation data will allow for a more robust calibration of the hydrology model leading to more refined design parameters.
- The cross-sectional data used in this level of effort analysis referred throughout Section 7 are solely based on cross-section data from 1979 (FEMA, 1991). Thus, the channel bottom used in this study is not representative of current conditions (2018). Consequently, it is highly recommended for the next level of analysis to conduct a topographic data collection campaign with the goal of properly mapping updated channel topography including left bank, main channel, and right bank.
- Research into historical flood marks along Dickinson Bayou is recommended to calibrate the hydraulic model.

- A field campaign for assessing Manning’s n is recommended for the next level of effort for Dickinson Bayou hydraulic analysis study to properly represent current conditions at the project site.

8.4 Galveston

In order to improve the interior drainage analysis for Galveston, new data collection is required. The following recommendations are suggested to help improve the future analysis:

- Install flow meters in the existing storm sewer system to collect real data and calibrate the SWMM model;
- Map existing storm drainage system within the Galveston watershed to include in the SWMM model;
- Obtain a higher resolved and more recent DEM of the Galveston watershed; and
- Once the seawall expansion/flood barrier designs are finalized, re-evaluate the impact of overtopping on the required pump and channel sizes. It is recommended that physical modeling be conducted to determine the final overtopping design volumes.
- Acquire additional utility information in the vicinity of the pumping station and consolidation conduits.

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Appendix A. Input Conditions for Overtopping Analysis

Site	Galveston
MM Extraction Point	1
USACE Extraction Point	9648
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
0.5	3.7	0.7	2.4	13.3	17	4	6E-69	15,689	0.0
1	3.7	0.7	2.4	13.3	17	4	6E-67	15,689	0.0
1.5	3.7	0.8	2.4	13.3	17	4	2E-64	15,689	0.0
2	3.8	0.8	2.4	13.2	17	4	5E-62	15,689	0.0
2.5	3.8	0.9	2.4	13.2	17	4	7E-57	15,689	0.0
3	3.9	0.9	2.4	13.1	17	4	2E-55	15,689	0.0
3.5	4.0	1.0	2.4	13.0	17	4	5E-53	15,689	0.0
4	4.1	1.0	2.7	12.9	17	4	2E-45	15,689	0.0
4.5	4.2	1.1	2.7	12.8	17	4	4E-43	15,689	0.0
5	4.3	1.2	2.7	12.7	17	4	3E-41	15,689	0.0
5.5	4.4	1.2	2.7	12.6	17	4	4E-39	15,689	0.0
6	4.5	1.3	2.7	12.5	17	4	9E-37	15,689	0.0
6.5	4.7	1.4	2.7	12.3	17	4	2E-34	15,689	0.0
7	4.8	1.7	2.7	12.2	17	4	2E-30	15,689	0.0
7.5	5.0	1.8	2.7	12.0	17	4	2E-28	15,689	0.0
8	5.3	2.0	2.9	11.7	17	4	4E-23	15,689	0.0
8.5	5.6	2.2	2.7	11.4	17	4	7E-24	15,689	0.0
9	5.9	2.5	2.7	11.1	17	4	4E-21	15,689	0.0
9.5	6.3	2.8	2.9	10.7	17	4	9E-17	15,689	0.0
10	6.8	3.1	2.9	10.2	17	4	1E-14	15,689	0.0
10.5	7.3	3.5	2.9	9.7	17	4	1E-12	15,689	0.0
11	7.8	3.9	2.9	9.2	17	4	6E-11	15,689	0.0
11.5	8.4	4.3	3.2	8.6	17	4	2E-08	15,689	0.0
12	8.9	4.7	3.2	8.1	17	4	2E-07	15,689	0.0
12.5	9.3	4.8	3.2	7.7	17	4	8E-07	15,689	0.0

Site	Galveston
MM Extraction Point	1
USACE Extraction Point	9648
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
13	9.4	4.4	2.9	7.6	17	4	5E-08	15,689	0.0
13.5	8.8	2.0	2.7	8.2	17	4	7E-17	15,689	0.0
14	7.3	0.9	2.4	9.7	17	4	1E-39	15,689	0.0
14.5	5.7	0.3	2.4	11.3	17	4	3E-177	15,689	0.0
15	4.9	0.2	2.4	12.1	17	4	7E-257	15,689	0.0
15.5	4.4	0.7	3.2	12.6	17	4	3E-55	15,689	0.0
16	4.3	1.0	3.6	12.7	17	4	4E-31	15,689	0.0
16.5	5.1	1.0	4.7	11.9	17	4	7E-31	15,689	0.0
17	5.8	1.4	4.3	11.2	17	4	5E-20	15,689	0.0
17.5	6.2	1.6	3.9	10.8	17	4	7E-17	15,689	0.0
18	6.1	1.8	3.6	10.9	17	4	1E-17	15,689	0.0
18.5	5.7	1.9	3.2	11.3	17	4	4E-20	15,689	0.0
19	5.2	1.7	2.9	11.8	17	4	2E-25	15,689	0.0
19.5	4.8	1.5	2.9	12.2	17	4	2E-29	15,689	0.0
20	4.3	1.4	2.7	12.7	17	4	9E-37	15,689	0.0
20.5	3.9	1.1	2.7	13.1	17	4	3E-43	15,689	0.0
21	3.5	1.0	2.7	13.5	17	4	4E-50	15,689	0.0
21.5	3.1	0.8	2.7	13.9	17	4	6E-59	15,689	0.0
22	2.8	0.8	2.4	14.2	17	4	6E-70	15,689	0.0
22.5	2.5	0.7	2.4	14.5	17	4	2E-79	15,689	0.0
23	2.3	0.6	2.4	14.7	17	4	4E-85	15,689	0.0
23.5	2.2	0.5	2.4	14.8	17	4	3E-92	15,689	0.0
24	2.3	0.5	2.4	14.7	17	4	1E-95	15,689	0.0
24.5	2.4	0.5	2.4	14.6	17	4	8E-96	15,689	0.0
25	2.7	0.5	2.4	14.3	17	4	6E-107	15,689	0.0

Site	Galveston
MM Extraction Point	1
USACE Extraction Point	9648
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
25.5	2.8	0.5	2.4	14.2	17	4	7E-108	15,689	0.0
26	3.0	0.4	2.4	14.0	17	4	2E-112	15,689	0.0
26.5	3.1	0.4	2.4	13.9	17	4	3E-116	15,689	0.0
27	3.2	0.4	2.4	13.8	17	4	3E-118	15,689	0.0
27.5	3.4	0.4	2.4	13.6	17	4	4E-120	15,689	0.0
28	3.5	0.4	2.4	13.5	17	4	4E-124	15,689	0.0
28.5	3.6	0.4	2.4	13.4	17	4	4E-131	15,689	0.0
29	3.7	0.4	2.4	13.3	17	4	2E-136	15,689	0.0
29.5	3.8	0.3	2.4	13.2	17	4	5E-143	15,689	0.0
30	3.8	0.3	2.4	13.2	17	4	3E-152	15,689	0.0
30.5	3.9	0.3	2.4	13.1	17	4	4E-161	15,689	0.0
31	4.0	0.3	2.4	13.0	17	4	5E-172	15,689	0.0

Site	Galveston
MM Extraction Point	2
USACE Extraction Point	8019
JPM-OS Storm Number	529

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
0.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
1	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
1.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
2	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
2.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
3	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
3.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
4	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
4.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
5.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
6	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
6.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
7	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
7.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
8	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
8.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
9	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
9.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
10	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
10.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
11	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
11.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
12	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
12.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0

Site	Galveston
MM Extraction Point	2
USACE Extraction Point	8019
JPM-OS Storm Number	529

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
13	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
13.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
14	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
14.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
15	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
15.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
16	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
16.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
17	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
17.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
18	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
18.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
19	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
19.5	0.0	0.0	0.0	17.0	17	4	0E+00	19,882	0.0
20	4.0	0.0	0.0	13.0	17	4	0E+00	19,882	0.0
20.5	4.3	0.0	0.0	12.7	17	4	0E+00	19,882	0.0
21	4.6	0.0	0.0	12.4	17	4	0E+00	19,882	0.0
21.5	4.9	0.0	0.0	12.1	17	4	0E+00	19,882	0.0
22	5.2	0.0	0.0	11.8	17	4	0E+00	19,882	0.0
22.5	5.5	0.1	3.9	11.5	17	4	3E-285	19,882	0.0
23	5.3	0.0	3.6	11.7	17	4	0E+00	19,882	0.0
23.5	4.3	0.0	2.7	12.7	17	4	0E+00	19,882	0.0
24	5.2	0.0	2.4	11.8	17	4	0E+00	19,882	0.0
24.5	7.4	0.4	2.4	9.6	17	4	1E-79	19,882	0.0
25	7.5	0.5	2.4	9.5	17	4	3E-65	19,882	0.0

Site	Galveston
MM Extraction Point	2
USACE Extraction Point	8019
JPM-OS Storm Number	529

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
25.5	6.9	0.9	3.2	10.1	17	4	2E-28	19,882	0.0
26	6.7	0.8	3.6	10.3	17	4	4E-36	19,882	0.0
26.5	6.9	0.9	3.9	10.1	17	4	3E-29	19,882	0.0
27	6.7	0.8	4.3	10.3	17	4	1E-34	19,882	0.0
27.5	6.5	0.7	3.9	10.5	17	4	2E-45	19,882	0.0
28	6.2	0.5	3.9	10.8	17	4	8E-75	19,882	0.0
28.5	5.6	0.1	3.6	11.4	17	4	5E-163	19,882	0.0
29	5.2	0.0	3.9	11.8	17	4	0E+00	19,882	0.0
29.5	5.0	0.0	3.6	12.0	17	4	0E+00	19,882	0.0
30	4.7	0.0	3.6	12.3	17	4	0E+00	19,882	0.0
30.5	4.5	0.0	3.6	12.5	17	4	0E+00	19,882	0.0
31	4.4	0.0	3.6	12.6	17	4	0E+00	19,882	0.0
31.5	4.2	0.0	3.2	12.8	17	4	0E+00	19,882	0.0
32	4.1	0.0	3.2	12.9	17	4	0E+00	19,882	0.0
32.5	3.9	0.0	3.2	13.1	17	4	0E+00	19,882	0.0
33	0.0	0.0	2.2	17.0	17	4	0E+00	19,882	0.0
33.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,882	0.0
34	0.0	0.0	2.2	17.0	17	4	0E+00	19,882	0.0
34.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,882	0.0
35	0.0	0.0	2.2	17.0	17	4	0E+00	19,882	0.0
35.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,882	0.0
36	0.0	0.0	2.2	17.0	17	4	0E+00	19,882	0.0
36.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,883	0.0
37	0.0	0.0	2.2	17.0	17	4	0E+00	19,884	0.0
37.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,885	0.0

Site	Galveston
MM Extraction Point	2
USACE Extraction Point	8019
JPM-OS Storm Number	529

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
38	0.0	0.0	2.2	17.0	17	4	0E+00	19,886	0.0
38.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,887	0.0
39	0.0	0.0	2.2	17.0	17	4	0E+00	19,888	0.0
39.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,889	0.0
40	0.0	0.0	2.2	17.0	17	4	0E+00	19,890	0.0
40.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,891	0.0
41	0.0	0.0	2.2	17.0	17	4	0E+00	19,892	0.0
41.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,893	0.0
42	0.0	0.0	2.2	17.0	17	4	0E+00	19,894	0.0
42.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,895	0.0
43	0.0	0.0	2.2	17.0	17	4	0E+00	19,896	0.0
43.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,897	0.0
44	0.0	0.0	2.2	17.0	17	4	0E+00	19,898	0.0
44.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,899	0.0
45	0.0	0.0	2.2	17.0	17	4	0E+00	19,900	0.0
45.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,901	0.0
46	0.0	0.0	2.2	17.0	17	4	0E+00	19,902	0.0
46.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,903	0.0
47	0.0	0.0	2.2	17.0	17	4	0E+00	19,904	0.0
47.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,905	0.0
48	0.0	0.0	2.2	17.0	17	4	0E+00	19,906	0.0
48.5	0.0	0.0	2.2	17.0	17	4	0E+00	19,907	0.0

Site	Galveston
MM Extraction Point	3
USACE Extraction Point	9654
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
0.5	3.3	0.8	2.4	13.7	17	4	7E-64	23,606	0.0
1	3.3	0.8	2.4	13.7	17	4	1E-63	23,606	0.0
1.5	3.3	0.8	2.4	13.7	17	4	1E-63	23,606	0.0
2	3.3	0.9	2.4	13.7	17	4	4E-62	23,606	0.0
2.5	3.3	0.9	2.4	13.7	17	4	6E-61	23,606	0.0
3	3.3	0.9	2.4	13.7	17	4	7E-62	23,606	0.0
3.5	3.4	0.9	2.4	13.6	17	4	7E-61	23,606	0.0
4	3.4	0.9	2.4	13.6	17	4	6E-59	23,606	0.0
4.5	3.4	0.9	2.4	13.6	17	4	3E-58	23,606	0.0
5	3.4	0.9	2.4	13.6	17	4	9E-58	23,606	0.0
5.5	3.5	0.9	2.4	13.5	17	4	3E-57	23,606	0.0
6	3.5	1.0	2.4	13.5	17	4	1E-56	23,606	0.0
6.5	3.5	1.0	2.4	13.5	17	4	3E-56	23,606	0.0
7	3.5	1.0	2.4	13.5	17	4	1E-55	23,606	0.0
7.5	3.6	1.0	2.4	13.4	17	4	4E-55	23,606	0.0
8	3.6	1.0	2.4	13.4	17	4	1E-54	23,606	0.0
8.5	3.7	0.9	2.4	13.3	17	4	4E-58	23,606	0.0
9	3.7	0.9	2.4	13.3	17	4	2E-57	23,606	0.0
9.5	3.8	0.9	2.4	13.2	17	4	6E-57	23,606	0.0
10	3.8	0.9	2.4	13.2	17	4	2E-56	23,606	0.0
10.5	3.9	0.9	2.4	13.1	17	4	9E-56	23,606	0.0
11	4.0	0.9	2.4	13.0	17	4	2E-55	23,606	0.0
11.5	4.1	0.9	2.4	12.9	17	4	9E-55	23,606	0.0
12	4.1	0.9	2.4	12.9	17	4	4E-54	23,606	0.0
12.5	4.2	0.9	2.4	12.8	17	4	2E-53	23,606	0.0

Site	Galveston
MM Extraction Point	3
USACE Extraction Point	9654
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
13	4.4	1.0	2.4	12.6	17	4	1E-51	23,606	0.0
13.5	4.5	1.1	2.4	12.5	17	4	3E-48	23,606	0.0
14	4.7	1.3	2.4	12.3	17	4	2E-42	23,606	0.0
14.5	4.9	1.4	2.4	12.1	17	4	1E-39	23,606	0.0
15	5.1	1.4	2.4	11.9	17	4	3E-38	23,606	0.0
15.5	5.3	1.5	2.4	11.7	17	4	4E-35	23,606	0.0
16	5.7	1.7	2.4	11.3	17	4	4E-31	23,606	0.0
16.5	6.1	1.9	2.4	10.9	17	4	9E-28	23,606	0.0
17	6.6	2.1	2.7	10.4	17	4	8E-22	23,606	0.0
17.5	7.2	2.3	2.7	9.8	17	4	3E-19	23,606	0.0
18	7.9	2.5	2.7	9.1	17	4	2E-16	23,606	0.0
18.5	8.8	2.7	2.7	8.2	17	4	1E-13	23,606	0.0
19	9.9	3.1	2.7	7.1	17	4	1E-10	23,606	0.0
19.5	11.1	2.6	2.7	5.9	17	4	3E-09	23,606	0.0
20	11.7	3.1	2.7	5.3	17	4	3E-07	23,606	0.0
20.5	10.7	1.9	2.4	6.3	17	4	5E-14	23,606	0.0
21	9.2	0.6	2.4	7.8	17	4	2E-39	23,606	0.0
21.5	8.1	0.5	2.4	8.9	17	4	1E-52	23,606	0.0
22	6.7	0.7	2.4	10.3	17	4	4E-48	23,606	0.0
22.5	5.2	1.0	2.4	11.8	17	4	9E-48	23,606	0.0
23	3.5	1.0	2.4	13.5	17	4	3E-55	23,606	0.0
23.5	2.6	0.5	2.4	14.4	17	4	3E-89	23,606	0.0
24	2.0	0.2	2.4	15.0	17	4	0E+00	23,606	0.0
24.5	1.5	0.0	2.4	15.5	17	4	0E+00	23,606	0.0
25	1.2	0.0	2.4	15.8	17	4	0E+00	23,606	0.0

Site	Galveston
MM Extraction Point	3
USACE Extraction Point	9654
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
25.5	1.1	0.0	2.4	15.9	17	4	0E+00	23,606	0.0
26	1.0	0.0	2.4	16.0	17	4	0E+00	23,606	0.0
26.5	1.0	0.0	2.4	16.0	17	4	0E+00	23,606	0.0
27	1.0	0.0	2.4	16.0	17	4	0E+00	23,606	0.0
27.5	1.1	0.0	2.4	15.9	17	4	0E+00	23,606	0.0
28	1.2	0.0	2.4	15.8	17	4	0E+00	23,606	0.0
28.5	1.4	0.0	2.4	15.6	17	4	0E+00	23,606	0.0
29	1.5	0.0	2.4	15.5	17	4	0E+00	23,606	0.0
29.5	1.7	0.0	2.4	15.3	17	4	0E+00	23,606	0.0
30	1.9	0.1	2.4	15.1	17	4	0E+00	23,606	0.0
30.5	2.1	0.2	2.4	14.9	17	4	0E+00	23,606	0.0
31	2.3	0.3	2.4	14.7	17	4	0E+00	23,606	0.0
31.5	2.5	0.4	2.4	14.5	17	4	0E+00	23,606	0.0
32	2.6	0.5	2.4	14.4	17	4	0E+00	23,606	0.0
32.5	2.7	0.6	2.4	14.3	17	4	0E+00	23,606	0.0
33	2.8	0.6	2.4	14.2	17	4	0E+00	23,606	0.0
33.5	2.9	0.7	2.4	14.1	17	4	0E+00	23,606	0.0
34	2.9	0.7	2.4	14.1	17	4	0E+00	23,606	0.0
34.5	3.0	0.7	2.4	14.0	17	4	0E+00	23,606	0.0
35	3.0	0.7	2.4	14.0	17	4	0E+00	23,606	0.0
35.5	3.0	0.7	2.4	14.0	17	4	0E+00	23,606	0.0
36	3.0	0.7	2.4	14.0	17	4	0E+00	23,606	0.0
36.5	3.1	0.7	2.4	13.9	17	4	0E+00	23,607	0.0
37	3.1	0.7	2.4	13.9	17	4	0E+00	23,608	0.0
37.5	3.1	0.7	2.4	13.9	17	4	0E+00	23,609	0.0

Site	Galveston
MM Extraction Point	3
USACE Extraction Point	9654
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Levee [ft NAVD88]	slope (H:V)	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
38	3.1	0.8	2.2	13.9	17	4	0E+00	23,610	0.0
38.5	3.1	0.8	2.2	13.9	17	4	0E+00	23,611	0.0
39	3.2	0.8	2.2	13.8	17	4	0E+00	23,612	0.0
39.5	3.2	0.8	2.2	13.8	17	4	0E+00	23,613	0.0
40	3.2	0.8	2.2	13.8	17	4	0E+00	23,614	0.0
40.5	3.2	0.8	2.2	13.8	17	4	0E+00	23,615	0.0

Site	Galveston
MM Extraction Point	4
USACE Extraction Point	7681
JPM-OS Storm Number	453

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	T.O. Seawall [ft NAVD88]	Rc [ft]	Ac [ft]	cot (a_d)	cot (a_u)	h [ft]	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
0.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
1	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
1.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
2	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
2.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
3	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
3.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
4	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
4.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
5.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
6	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
6.5	5.6	0.0	0.0	21	15.4	8.9	0.955	0.955	0.00	0.00	42,414	0
7	5.7	0.0	0.0	21	15.3	8.8	0.955	0.955	0.00	0.00	42,414	0
7.5	6.1	0.2	11.2	21	14.9	8.4	0.955	0.955	0.28	0.02	42,414	639
8	6.4	0.4	11.2	21	14.6	8.1	0.955	0.955	0.63	0.02	42,414	776
8.5	6.8	0.6	11.2	21	14.2	7.7	0.955	0.955	1.03	0.01	42,414	236
9	7.1	0.8	11.2	21	13.9	7.4	0.955	0.955	1.36	0.00	42,414	137
9.5	7.5	1.1	11.2	21	13.5	7.0	0.955	0.955	1.68	0.00	42,414	114
10	7.6	1.2	11.2	21	13.4	6.9	0.955	0.955	1.87	0.00	42,414	82
10.5	8.1	1.4	10.2	21	12.9	6.4	0.955	0.955	2.31	0.00	42,414	56
11	8.8	1.9	10.2	21	12.2	5.7	0.955	0.955	3.03	0.00	42,414	68
11.5	9.4	2.3	10.2	21	11.6	5.1	0.955	0.955	3.66	0.00	42,414	126
12	10.6	3.0	10.2	21	10.4	3.9	0.955	0.955	4.82	0.01	42,414	245

Site	Galveston
MM Extraction Point	4
USACE Extraction Point	7681
JPM-OS Storm Number	453

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	T.O. Seawall [ft NAVD88]	Rc [ft]	Ac [ft]	cot (a_d)	cot (a_u)	h [ft]	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
12.5	11.9	3.8	10.2	21	9.1	2.6	0.955	0.955	6.16	0.01	42,414	621
13	13.7	4.9	10.2	21	7.3	0.8	0.955	0.955	7.98	0.14	42,414	6,072
13.5	16.0	6.2	10.2	21	5.0	-1.5	0.955	0.955	10.21	1.29	42,414	54,785
14	17.7	7.1	9.2	21	3.3	-3.2	0.955	0.955	11.89	3.49	42,414	147,919
14.5	16.4	6.3	7.6	21	4.6	-1.9	0.955	0.955	10.61	1.37	42,414	57,981
15	14.0	5.0	8.4	21	7.0	0.5	0.955	0.955	8.27	0.16	42,414	6,939
15.5	12.7	4.2	8.4	21	8.3	1.8	0.955	0.955	6.88	0.03	42,414	1,146
16	11.8	3.6	7.6	21	9.2	2.7	0.955	0.955	6.01	0.01	42,414	373
16.5	11.0	3.2	7.6	21	10.0	3.5	0.955	0.955	5.19	0.00	42,414	209
17	9.7	2.4	7.6	21	11.3	4.8	0.955	0.955	3.92	0.00	42,414	116
17.5	8.4	1.7	8.4	21	12.6	6.1	0.955	0.955	2.67	0.00	42,414	50
18	7.4	1.0	7.6	21	13.6	7.1	0.955	0.955	1.64	0.00	42,414	117
18.5	6.5	0.5	7.6	21	14.5	8.0	0.955	0.955	0.76	0.01	42,414	383
19	5.9	0.1	2.4	21	15.1	8.6	0.955	0.955	0.12	0.01	42,414	424
19.5	5.5	0.0	0.0	21	15.5	9.0	0.955	0.955	0.00	0.00	42,414	0
20	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
20.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
21	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
21.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
22	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
22.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
23	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
23.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
24	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0

Site	Galveston
MM Extraction Point	4
USACE Extraction Point	7681
JPM-OS Storm Number	453

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	T.O. Seawall [ft NAVD88]	Rc [ft]	Ac [ft]	cot (a_d)	cot (a_u)	h [ft]	q [cfs/ft]	Shoreline Length [ft]	Q [cfs]
24.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
25	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
25.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
26	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
26.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
27	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
27.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
28	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
28.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
29	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
29.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
30	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
30.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
31	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
31.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
32	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
32.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
33	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
33.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
34	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
34.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
35	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
35.5	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0
36	0.0	0.0	0.0	21	21.0	14.5	0.955	0.955	0.00	0.00	42,414	0

Site	Clear Creek
MM Extraction Point	--
USACE Extraction Point	122
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
0.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
1	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
1.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
2	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
2.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
3	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
3.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
4	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
4.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
5.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
6	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
6.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
7	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
7.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
8	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
8.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
9	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
9.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
10	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
10.5	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
11	0.0	0.0	0.0	17.0	17.0	0.00	2985	0
11.5	7.6	1.7	4.7	9.4	17.0	0.00	2985	0
12	8.3	2.0	4.7	8.7	17.0	0.00	2985	0
12.5	9.1	3.0	5.2	7.9	17.0	0.00	2985	6

Site	Clear Creek
MM Extraction Point	--
USACE Extraction Point	122
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
13	10.2	3.4	5.7	6.8	17.0	0.01	2985	32
13.5	11.8	3.9	6.3	5.2	17.0	0.09	2985	255
14	13.3	4.5	6.3	3.7	17.0	0.39	2985	1161
14.5	12.9	3.5	6.9	4.1	17.0	0.11	2985	329
15	10.3	2.3	6.3	6.7	17.0	0.00	2985	1
15.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
16	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
16.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
17	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
17.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
18	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
18.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
19	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
19.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
20	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
20.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
21	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
21.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
22	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
22.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
23	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
23.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
24	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
24.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
25	0.0	0.0	2.2	17.0	17.0	0.00	2985	0

Site	Clear Creek
MM Extraction Point	--
USACE Extraction Point	122
JPM-OS Storm Number	356

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
25.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
26	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
26.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
27	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
27.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
28	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
28.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
29	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
29.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
30	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
30.5	0.0	0.0	2.2	17.0	17.0	0.00	2985	0
31	0.0	0.0	2.2	17.0	17.0	0.00	2985	0

Site	D. Bayou
MM Extraction Point	--
USACE Extraction Point	8776
JPM-OS Storm Number	633

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
0.5	3.6	0.5	3.2	14.4	18.0	0.00	879	0
1	3.6	0.5	3.2	14.4	18.0	0.00	879	0
1.5	3.6	0.5	3.2	14.4	18.0	0.00	879	0
2	3.6	0.5	3.2	14.4	18.0	0.00	879	0
2.5	3.6	0.5	3.2	14.4	18.0	0.00	879	0
3	3.7	0.5	3.6	14.3	18.0	0.00	879	0
3.5	3.7	0.5	3.6	14.3	18.0	0.00	879	0
4	3.7	0.5	3.6	14.3	18.0	0.00	879	0
4.5	3.7	0.5	3.2	14.3	18.0	0.00	879	0
5	3.7	0.5	3.2	14.3	18.0	0.00	879	0
5.5	3.7	0.5	3.2	14.3	18.0	0.00	879	0
6	3.7	0.5	3.2	14.3	18.0	0.00	879	0
6.5	3.7	0.5	3.2	14.3	18.0	0.00	879	0
7	3.7	0.5	3.2	14.3	18.0	0.00	879	0
7.5	3.8	0.5	3.6	14.2	18.0	0.00	879	0
8	3.8	0.5	3.6	14.2	18.0	0.00	879	0
8.5	3.8	0.5	3.6	14.2	18.0	0.00	879	0
9	3.8	0.5	3.6	14.2	18.0	0.00	879	0
9.5	3.9	0.5	3.6	14.1	18.0	0.00	879	0
10	3.9	0.6	3.6	14.1	18.0	0.00	879	0
10.5	3.9	0.6	3.6	14.1	18.0	0.00	879	0
11	3.9	0.6	3.6	14.1	18.0	0.00	879	0
11.5	4.0	0.6	3.6	14.0	18.0	0.00	879	0
12	4.0	0.6	3.6	14.0	18.0	0.00	879	0
12.5	4.0	0.6	3.6	14.0	18.0	0.00	879	0

Site	D. Bayou
MM Extraction Point	--
USACE Extraction Point	8776
JPM-OS Storm Number	633

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
13	4.0	0.6	3.6	14.0	18.0	0.00	879	0
13.5	4.0	0.6	3.6	14.0	18.0	0.00	879	0
14	4.1	0.7	3.6	13.9	18.0	0.00	879	0
14.5	4.1	0.7	3.6	13.9	18.0	0.00	879	0
15	4.2	0.7	3.6	13.8	18.0	0.00	879	0
15.5	4.2	0.7	3.6	13.8	18.0	0.00	879	0
16	4.3	0.7	3.6	13.7	18.0	0.00	879	0
16.5	4.3	0.7	3.9	13.7	18.0	0.00	879	0
17	4.4	0.7	3.9	13.6	18.0	0.00	879	0
17.5	4.4	0.8	3.9	13.6	18.0	0.00	879	0
18	4.5	0.8	3.9	13.5	18.0	0.00	879	0
18.5	4.5	0.8	3.9	13.5	18.0	0.00	879	0
19	4.5	0.8	3.9	13.5	18.0	0.00	879	0
19.5	4.5	0.8	3.9	13.5	18.0	0.00	879	0
20	4.6	0.9	3.9	13.4	18.0	0.00	879	0
20.5	4.7	0.9	3.9	13.3	18.0	0.00	879	0
21	4.9	1.0	3.9	13.1	18.0	0.00	879	0
21.5	5.0	1.0	3.9	13.0	18.0	0.00	879	0
22	5.1	1.0	4.3	12.9	18.0	0.00	879	0
22.5	5.2	1.1	4.3	12.8	18.0	0.00	879	0
23	5.4	1.2	4.3	12.6	18.0	0.00	879	0
23.5	5.5	1.4	4.3	12.5	18.0	0.00	879	0
24	5.7	1.4	4.3	12.3	18.0	0.00	879	0
24.5	5.9	1.5	2.4	12.1	18.0	0.00	879	0
25	6.1	1.7	2.4	11.9	18.0	0.00	879	0

Site	D. Bayou
MM Extraction Point	--
USACE Extraction Point	8776
JPM-OS Storm Number	633

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
25.5	6.3	1.9	2.7	11.7	18.0	0.00	879	0
26	6.6	2.1	2.9	11.4	18.0	0.00	879	0
26.5	6.9	2.4	2.9	11.1	18.0	0.00	879	0
27	7.3	2.7	3.2	10.7	18.0	0.00	879	0
27.5	7.6	2.9	3.2	10.4	18.0	0.00	879	0
28	8.0	3.1	3.2	10.0	18.0	0.00	879	0
28.5	8.4	3.5	3.2	9.6	18.0	0.00	879	1
29	8.8	3.7	3.2	9.2	18.0	0.00	879	4
29.5	9.3	4.0	3.2	8.7	18.0	0.01	879	9
30	9.7	4.3	3.2	8.3	18.0	0.02	879	17
30.5	10.1	4.4	3.6	7.9	18.0	0.03	879	26
31	10.5	4.6	3.6	7.5	18.0	0.05	879	42
31.5	10.9	4.7	3.6	7.1	18.0	0.07	879	61
32	11.3	5.1	3.6	6.7	18.0	0.13	879	112
32.5	11.7	5.2	3.6	6.3	18.0	0.18	879	158
33	12.0	5.5	3.6	6.0	18.0	0.26	879	229
33.5	12.4	5.6	3.6	5.6	18.0	0.34	879	302
34	12.7	5.7	3.6	5.3	18.0	0.43	879	378
34.5	12.8	5.8	3.6	5.2	18.0	0.48	879	425
35	12.8	5.8	3.6	5.2	18.0	0.48	879	419
35.5	12.5	5.7	3.6	5.5	18.0	0.39	879	346
36	12.0	5.4	3.6	6.0	18.0	0.24	879	210
36.5	11.3	5.0	3.2	6.7	18.0	0.12	879	110
37	10.6	4.7	2.9	7.4	18.0	0.06	879	49
37.5	9.7	4.1	2.9	8.3	18.0	0.01	879	12

Site	D. Bayou
MM Extraction Point	--
USACE Extraction Point	8776
JPM-OS Storm Number	633

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
38	8.8	3.5	2.7	9.2	18.0	0.00	879	2
38.5	7.7	3.1	2.7	10.3	18.0	0.00	879	0
39	6.5	2.5	2.7	11.5	18.0	0.00	879	0
39.5	5.6	1.8	2.7	12.4	18.0	0.00	879	0
40	4.8	1.5	2.7	13.2	18.0	0.00	879	0
40.5	4.1	1.3	2.7	13.9	18.0	0.00	879	0
41	3.7	1.7	2.7	14.3	18.0	0.00	879	0
41.5	3.2	1.5	2.7	14.8	18.0	0.00	879	0
42	2.8	1.4	2.7	15.2	18.0	0.00	879	0
42.5	2.5	1.3	2.7	15.5	18.0	0.00	879	0
43	2.4	1.3	2.7	15.6	18.0	0.00	879	0
43.5	2.2	1.2	2.7	15.8	18.0	0.00	879	0
44	2.2	1.2	2.7	15.8	18.0	0.00	879	0
44.5	2.4	1.2	2.7	15.6	18.0	0.00	879	0
45	2.7	1.2	2.7	15.3	18.0	0.00	879	0
45.5	3.2	1.1	2.4	14.8	18.0	0.00	879	0
46	3.5	1.1	2.4	14.5	18.0	0.00	879	0
46.5	3.5	1.0	2.4	14.5	18.0	0.00	879	0
47	3.5	1.0	2.4	14.5	18.0	0.00	879	0
47.5	3.5	0.9	2.4	14.5	18.0	0.00	879	0
48	3.5	0.9	2.4	14.5	18.0	0.00	879	0
48.5	3.5	0.8	2.4	14.5	18.0	0.00	879	0
49	3.5	0.8	2.4	14.5	18.0	0.00	879	0
49.5	3.5	0.8	2.4	14.5	18.0	0.00	879	0
50	3.5	0.7	2.4	14.5	18.0	0.00	879	0

Site	D. Bayou
MM Extraction Point	--
USACE Extraction Point	8776
JPM-OS Storm Number	633

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
50.5	3.5	0.7	2.4	14.5	18.0	0.00	879	0
51	3.4	0.7	2.4	14.6	18.0	0.00	879	0
51.5	3.4	0.6	2.4	14.6	18.0	0.00	879	0
52	3.5	0.6	2.4	14.5	18.0	0.00	879	0
52.5	3.5	0.6	2.4	14.5	18.0	0.00	879	0
53	3.5	0.6	2.4	14.5	18.0	0.00	879	0
53.5	3.5	0.6	2.4	14.5	18.0	0.00	879	0
54	3.5	0.6	2.4	14.5	18.0	0.00	879	0
54.5	3.6	0.5	2.4	14.4	18.0	0.00	879	0
55	3.6	0.5	2.4	14.4	18.0	0.00	879	0
55.5	3.7	0.5	2.4	14.3	18.0	0.00	879	0
56	3.7	0.5	2.4	14.3	18.0	0.00	879	0
56.5	3.8	0.5	2.4	14.2	18.0	0.00	879	0
57	3.8	0.5	2.4	14.2	18.0	0.00	879	0
57.5	3.9	0.5	2.4	14.1	18.0	0.00	879	0
58	3.9	0.5	2.4	14.1	18.0	0.00	879	0
58.5	4.0	0.4	2.4	14.0	18.0	0.00	879	0
59	4.0	0.4	2.4	14.0	18.0	0.00	879	0
59.5	4.1	0.4	2.4	13.9	18.0	0.00	879	0
60	4.1	0.4	2.4	13.9	18.0	0.00	879	0
60.5	4.2	0.4	2.4	13.8	18.0	0.00	879	0
61	4.2	0.4	2.4	13.8	18.0	0.00	879	0
61.5	4.2	0.4	2.4	13.8	18.0	0.00	879	0
62	4.2	0.4	2.4	13.8	18.0	0.00	879	0
62.5	4.3	0.4	2.4	13.7	18.0	0.00	879	0

Site	D. Bayou
MM Extraction Point	--
USACE Extraction Point	8776
JPM-OS Storm Number	633

Timestep [hrs]	WSE [ft NAVD 88]	Hs [ft]	Tm-1,0 [s]	Rc [ft]	T.O. Wall [ft NAVD88]	q [cfs/ft]	Ref. Shoreline [ft]	Q [cfs]
63	4.3	0.4	2.4	13.7	18.0	0.00	879	0
63.5	4.3	0.3	2.4	13.7	18.0	0.00	879	0
64	4.4	0.3	2.4	13.6	18.0	0.00	879	0
64.5	4.4	0.3	2.4	13.6	18.0	0.00	879	0
65	4.4	0.3	2.4	13.6	18.0	0.00	879	0
65.5	4.4	0.3	2.4	13.6	18.0	0.00	879	0
66	4.5	0.3	2.4	13.5	18.0	0.00	879	0
66.5	4.5	0.3	2.4	13.5	18.0	0.00	879	0
67	4.5	0.3	2.4	13.5	18.0	0.00	879	0
67.5	4.5	0.3	2.4	13.5	18.0	0.00	879	0
68	4.5	0.3	2.4	13.5	18.0	0.00	879	0
68.5	4.5	0.3	2.4	13.5	18.0	0.00	879	0
69	4.5	0.3	2.4	13.5	18.0	0.00	879	0
69.5	4.5	0.2	2.4	13.5	18.0	0.00	879	0

Appendix B. Supplemental Model Results for Clear Creek, Dickinson Bayou, and Galveston

B.1 Clear Creek

The following figures are supplement the results reported in Section 5.4. They document the peak water levels for the performance runs and for the pump capacity sizing for different return periods.

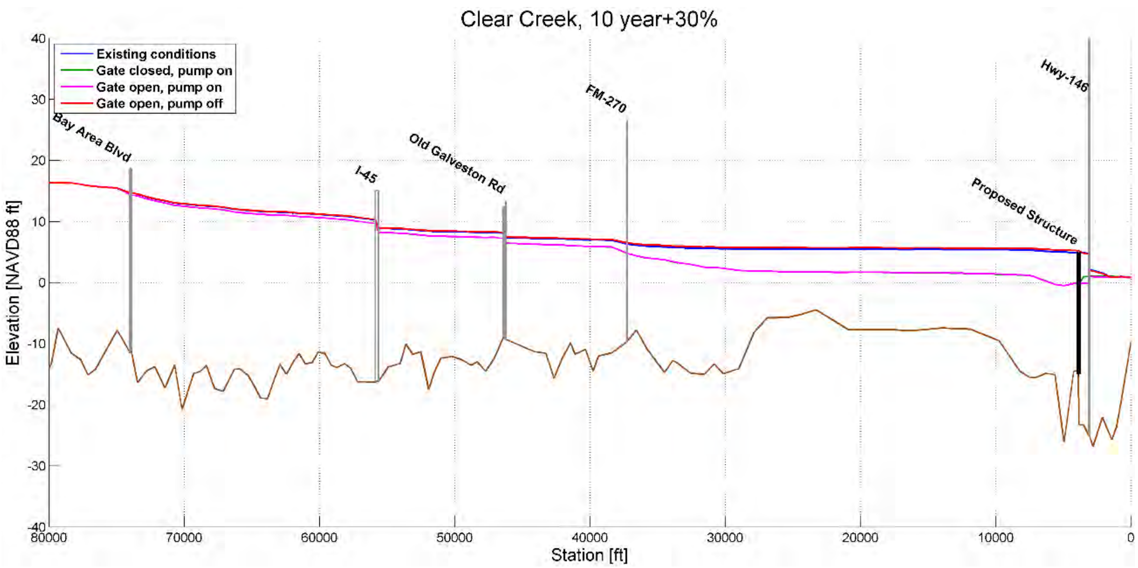


Figure B 1. Performance simulation of the 10-year (+30%) rainfall event on the Clear Creek watershed.

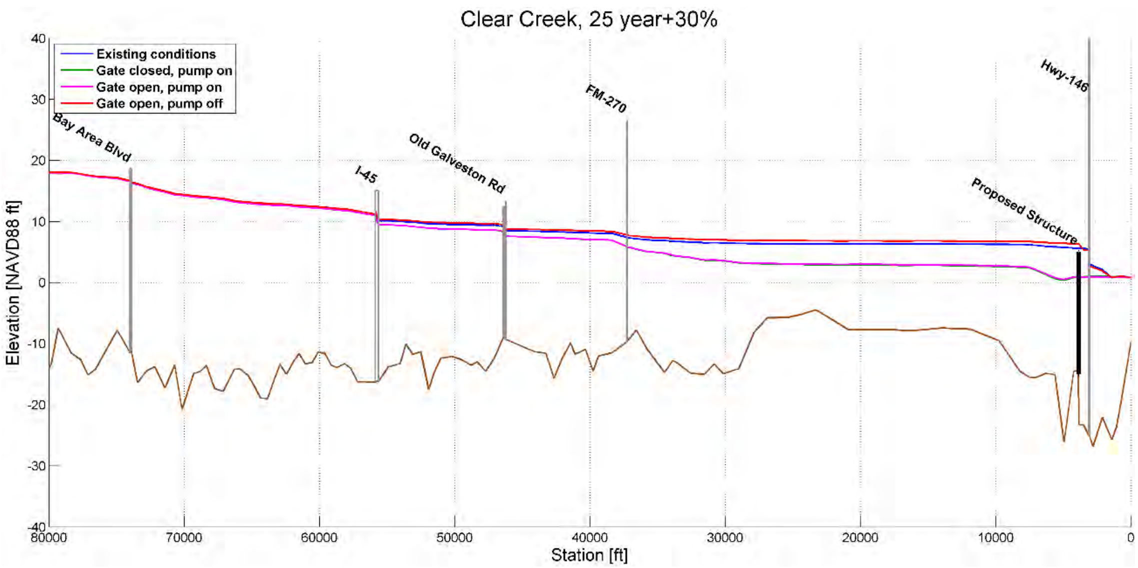


Figure B 2. Performance simulation of the 25-year (+30%) rainfall event on the Clear Creek watershed.

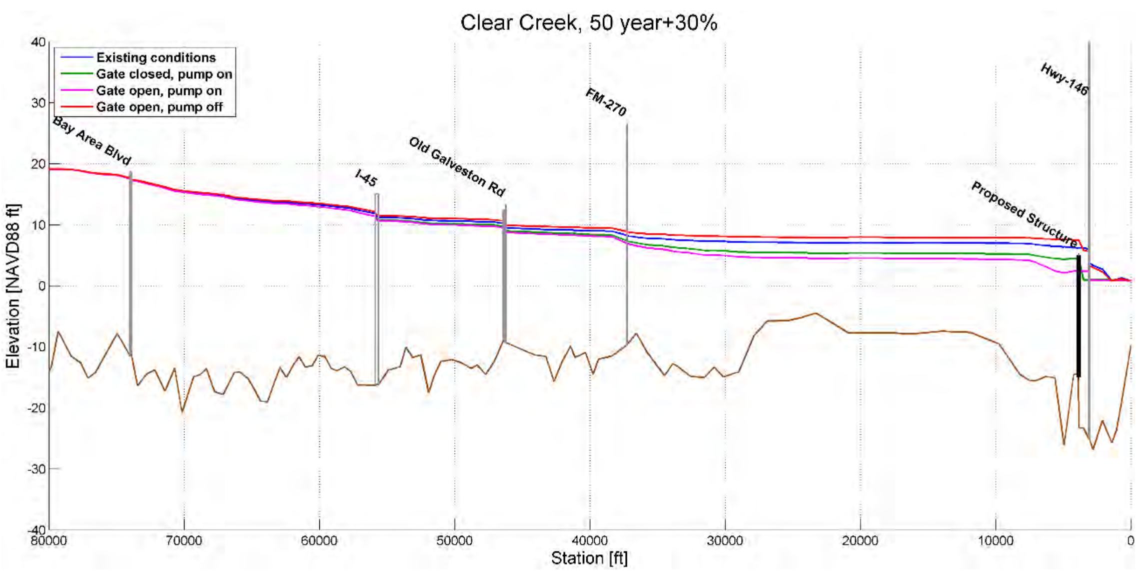


Figure B 3. Performance simulation of the 50-year (+30%) rainfall event on the Clear Creek watershed.

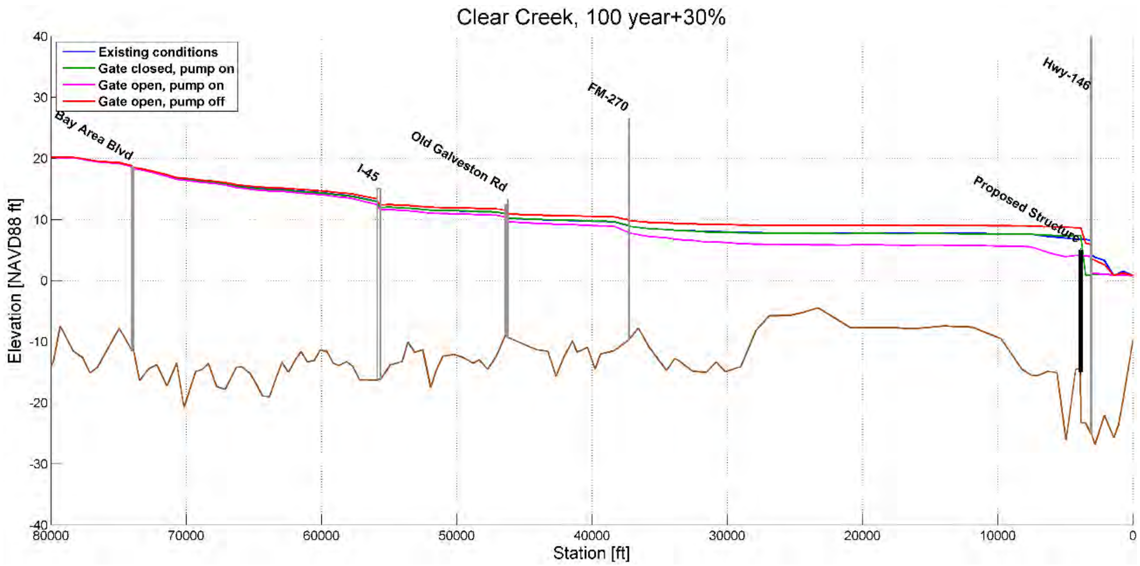


Figure B 4. Performance simulation of the 100-year (+30%) rainfall event on the Clear Creek watershed.

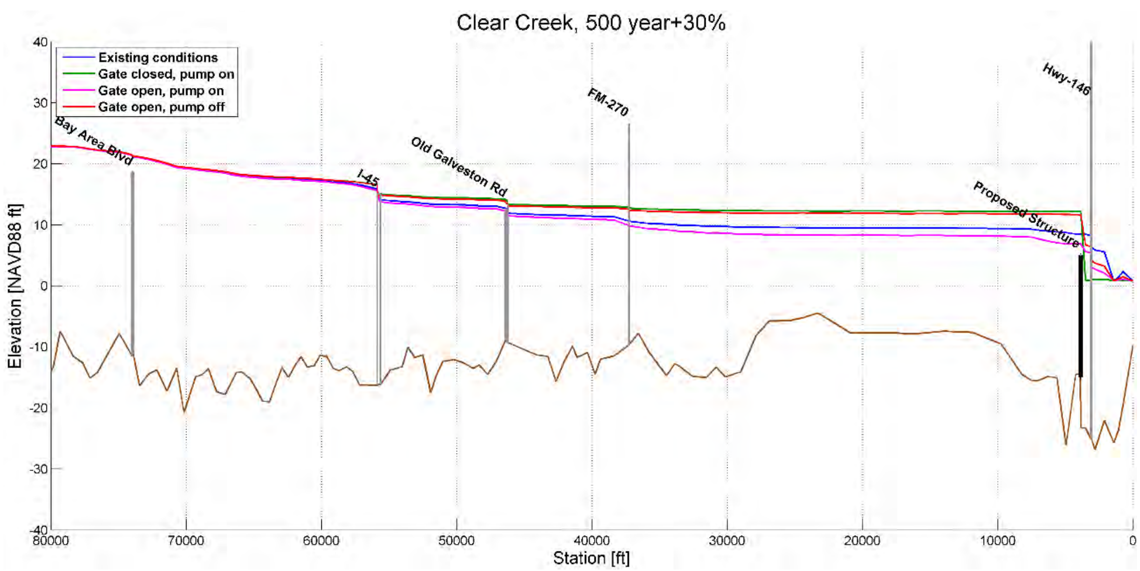


Figure B 5. Performance simulation of the 500-year (+30%) rainfall event on the Clear Creek watershed.

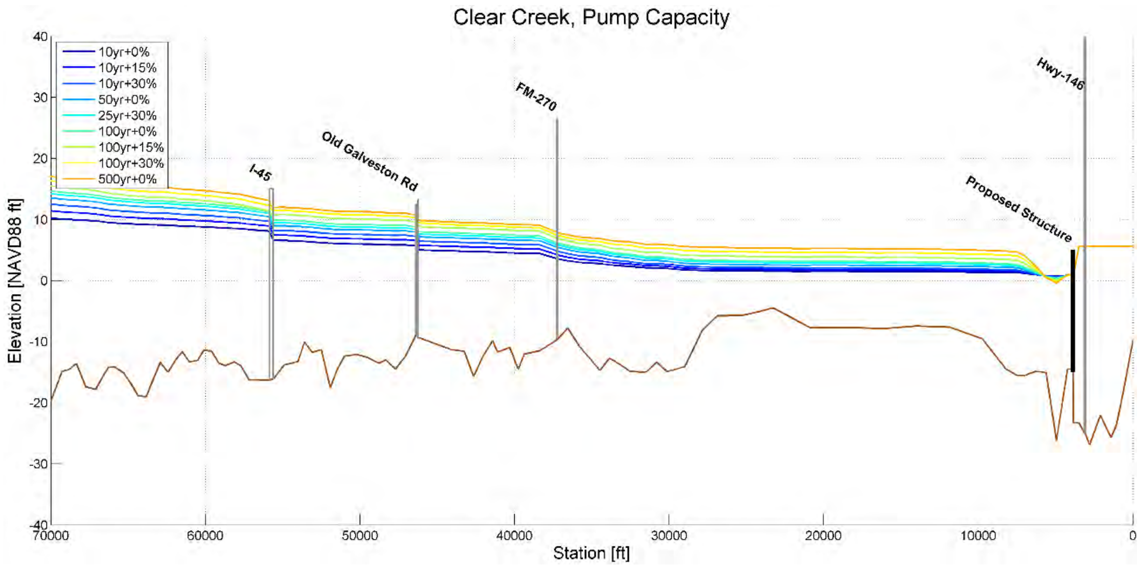


Figure B 6. Peak water level for pump capacity calibration simulations.

B.2 Dickinson Bayou

The following figures are supplement the results reported in Section 6.4. They document the peak water levels for the performance runs and for the pump capacity sizing for different return periods.

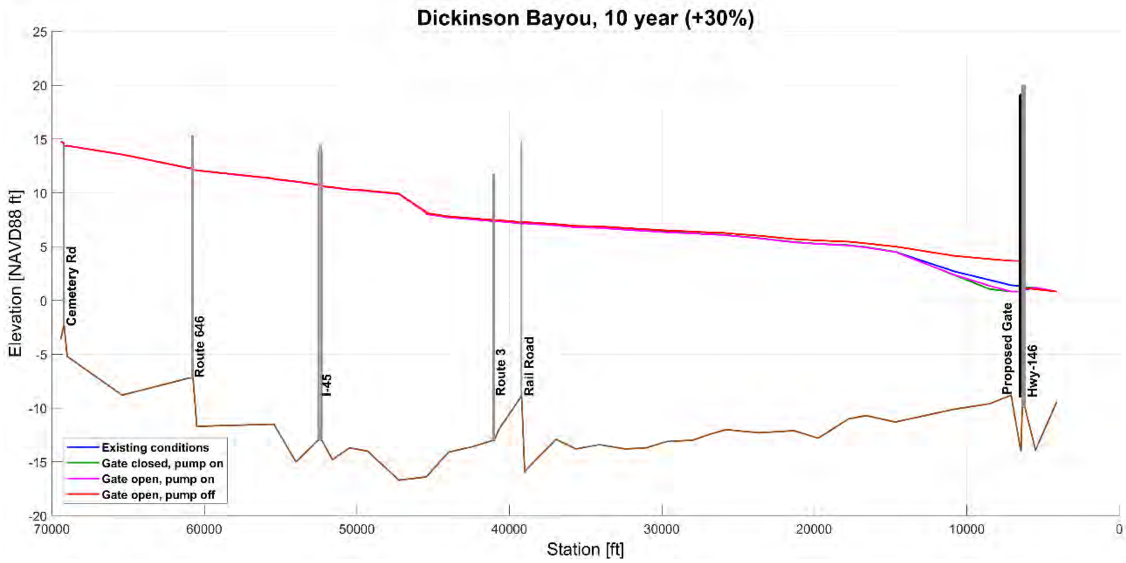


Figure B 7. Performance simulation of the 10-year (+30%) rainfall event on Dickinson Bayou.

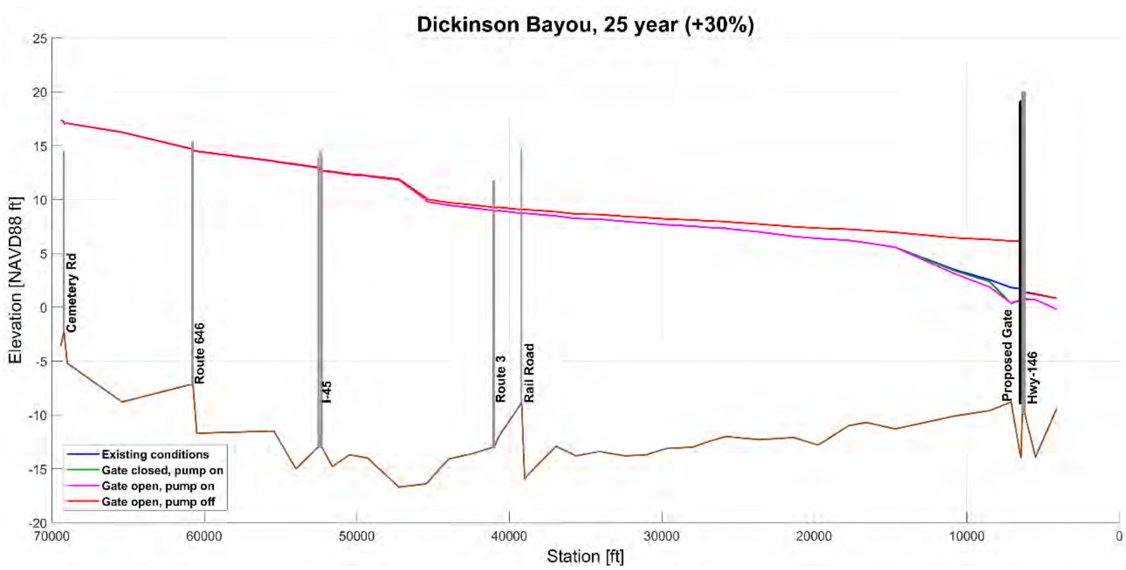


Figure B 8. Performance simulation of the 25-year (+30%) rainfall event on Dickinson Bayou.

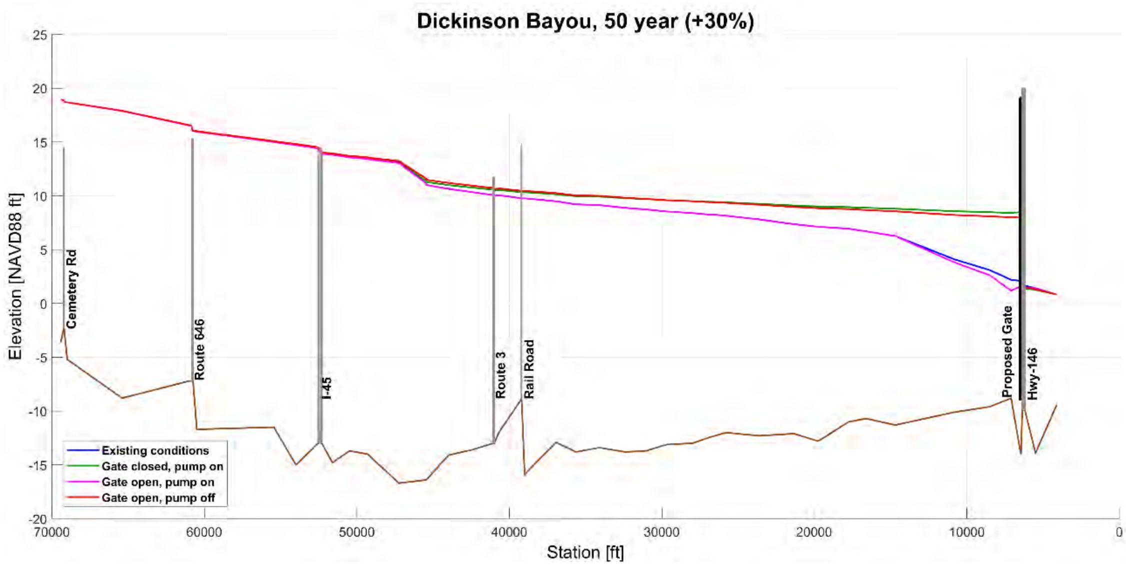


Figure B 9. Performance simulation of the 50-year (+30%) rainfall event on Dickinson Bayou.

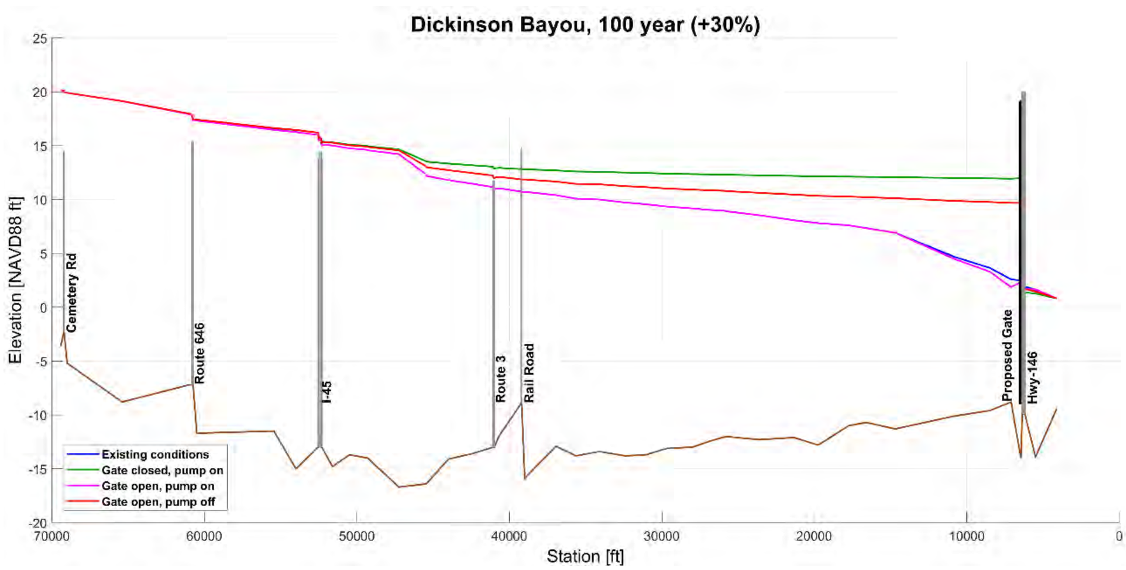


Figure B 10. Performance simulation of the 100-year (+30%) rainfall event on Dickinson Bayou.

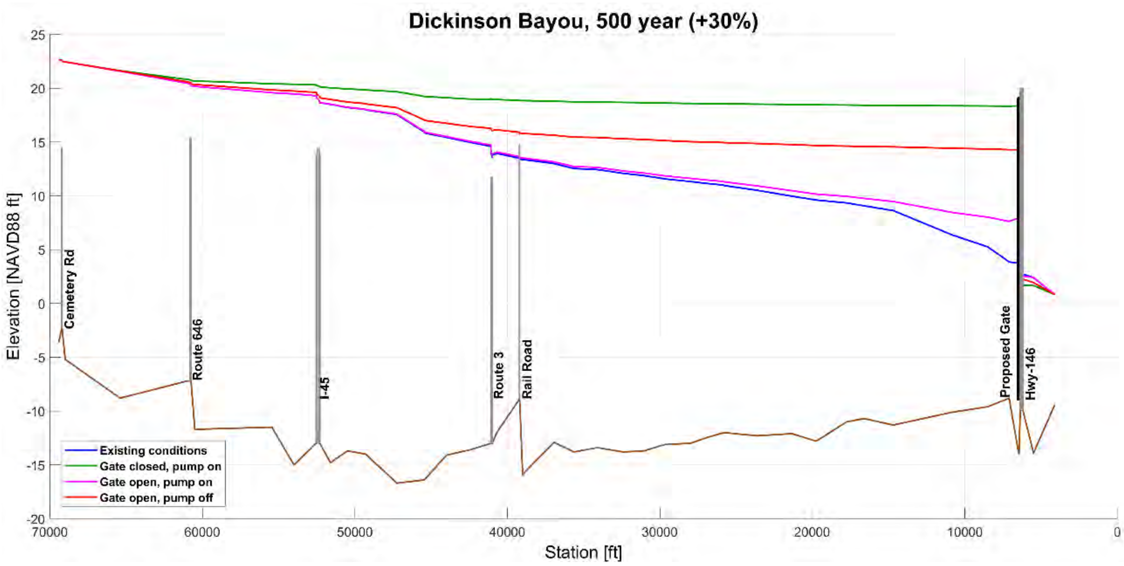


Figure B 11. Performance simulation of the 500-year (+30%) rainfall event on Dickinson Bayou.

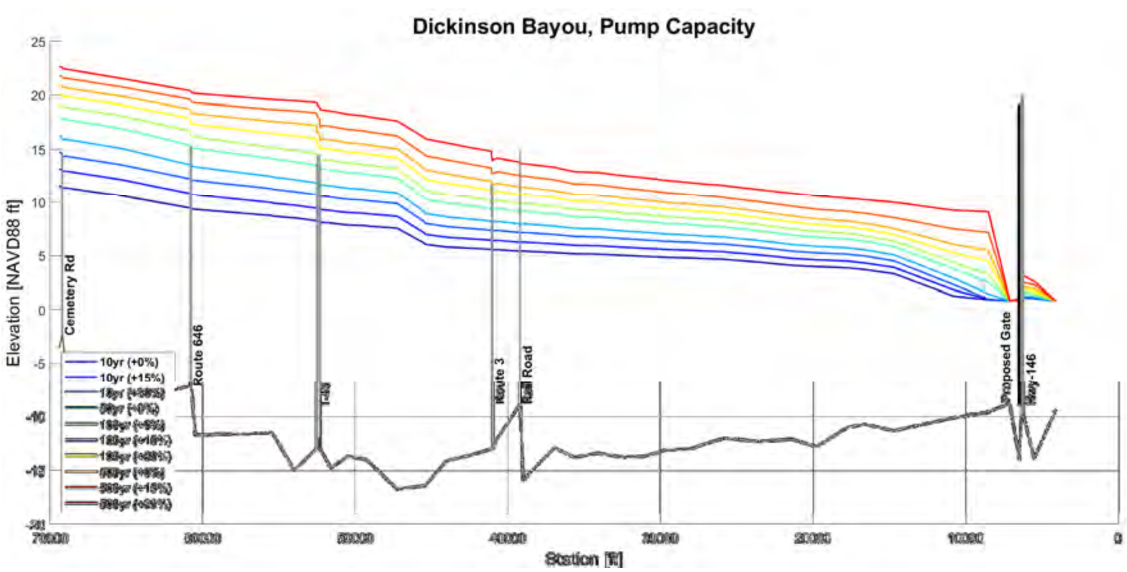


Figure B 12. Peak water level for different pump capacity at Dickinson Bayou.

B.3 Galveston

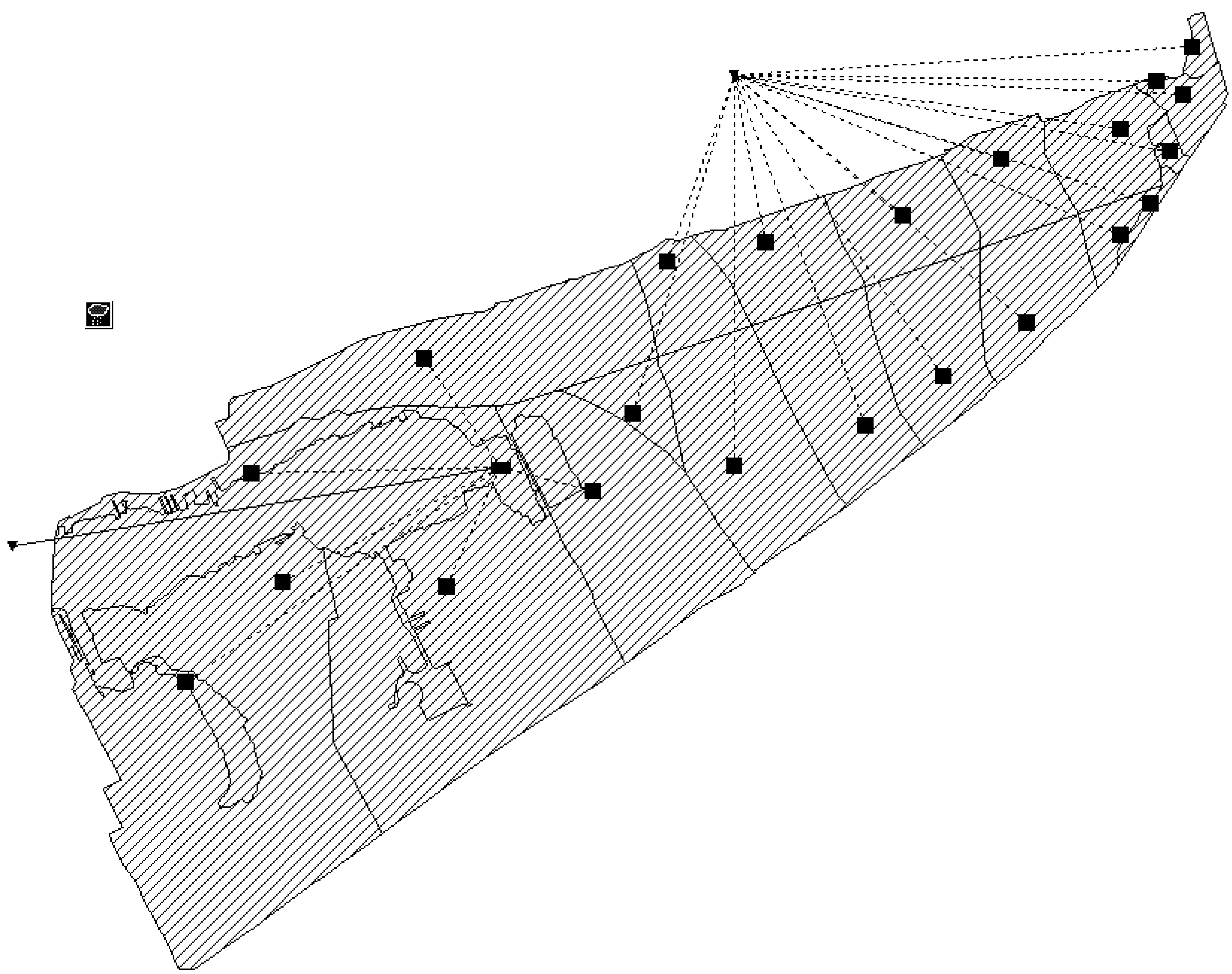
This section includes plots showing EPA SWMM outputs for water surface elevation profiles in the channels and Offatts Bayou, and flood volumes for the runs listed in the table below. All elevations shown within this section are in ft NAVD88.

RUN	MODEL CONFIGURATION	RAINFALL EVENT
1	Existing Conditions	10-yr+30%
2	Existing Conditions	25-yr+30%
3	Existing Conditions	50-yr+30%
4	Existing Conditions	100-yr+30%
5	Existing Conditions	500-yr+30%
6	Design Development for gates closed and pumps on	25-yr+30%
7	Proposed design facilities: gates closed, pumps on	10-yr+30%
8	Proposed design facilities: gates closed, pumps on	50-yr+30%
9	Proposed design facilities: gates closed, pumps on	100-yr+30%
10	Proposed design facilities: gates closed, pumps on	500-yr+30%
11	Proposed design facilities: gates open, pumps on	10-yr+30%
12	Proposed design facilities: gates open, pumps on	25-yr+30%
13	Proposed design facilities: gates open, pumps on	50-yr+30%
14	Proposed design facilities: gates open, pumps on	100-yr+30%
15	Proposed design facilities: gates open, pumps on	500-yr+30%
16	Proposed design facilities: gates open, pumps off	10-yr+30%
17	Proposed design facilities: gates open, pumps off	25-yr+30%
18	Proposed design facilities: gates open, pumps off	50-yr+30%
19	Proposed design facilities: gates open, pumps off	100-yr+30%
20	Proposed design facilities: gates open, pumps off	500-yr+30%
21	Proposed design conditions: gates open, tidal run	N/A
22	Proposed design conditions: gates closed, pumps on	Harvey
23	Alternative facility design development with gates closed, pumps on	10-yr+0%
24	Alternative facility design development with gates closed, pumps on	10-yr+15%
25	Alternative facility design development with gates closed, pumps on	10-yr+30%
26	Alternative facility design development with gates closed, pumps on	25-yr+0%
27	Alternative facility design development with gates closed, pumps on	25-yr+15%
28	Alternative facility design development with gates closed, pumps on	50-yr+0%
29	Alternative facility design development with gates closed, pumps on	50-yr+15%

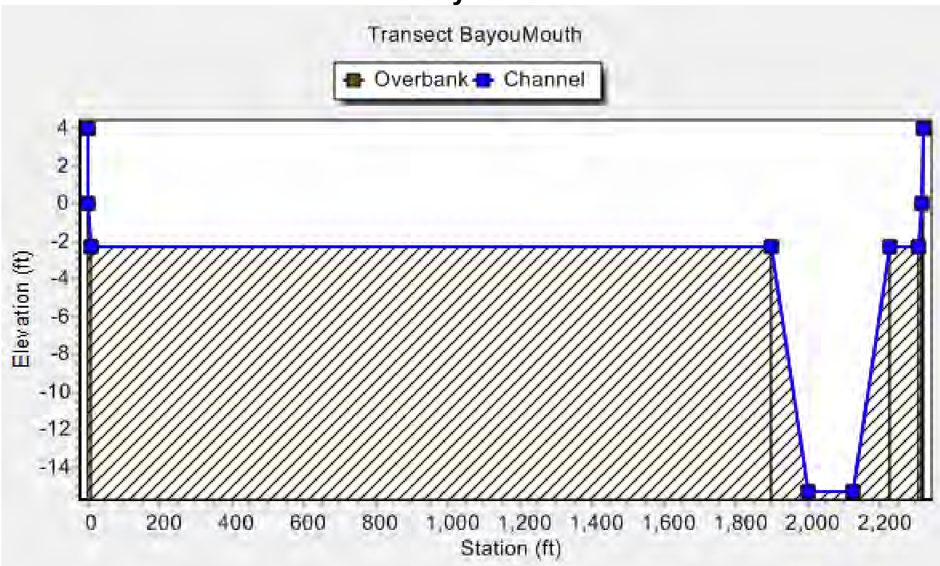
30	Alternative facility design development with gates closed, pumps on	50-yr+30%
31	Alternative facility design development with gates closed, pumps on	100-yr+0%
32	Alternative facility design development with gates closed, pumps on	100-yr+15%
33	Alternative facility design development with gates closed, pumps on	100-yr +30%
34	Alternative facility design development with gates closed, pumps on	500-yr+0%
35	Alternative facility design development with gates closed, pumps on	500-yr+15%
36	Alternative facility design development with gates closed, pumps on	500-yr+30%

Runs 1 – 5: Existing Conditions

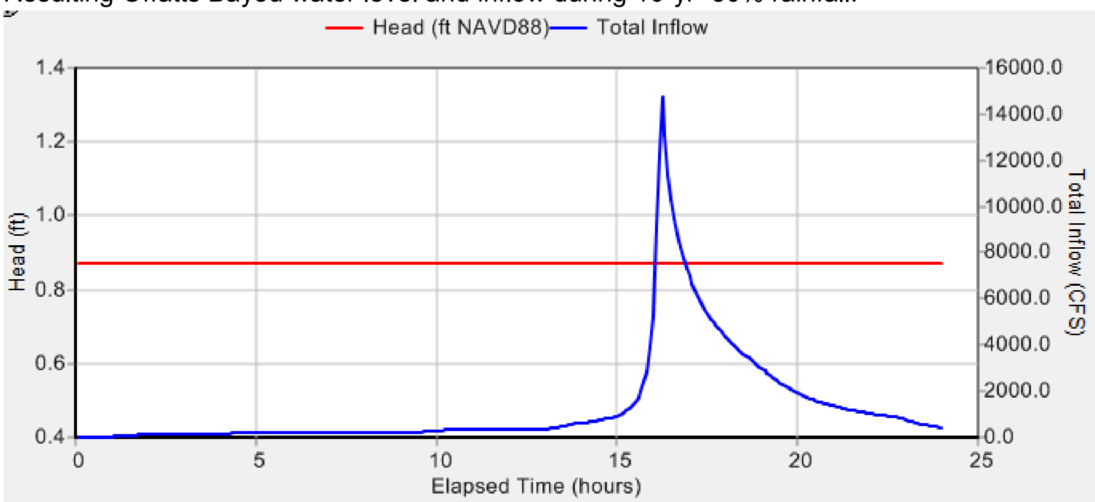
Galveston SWMM model configuration:



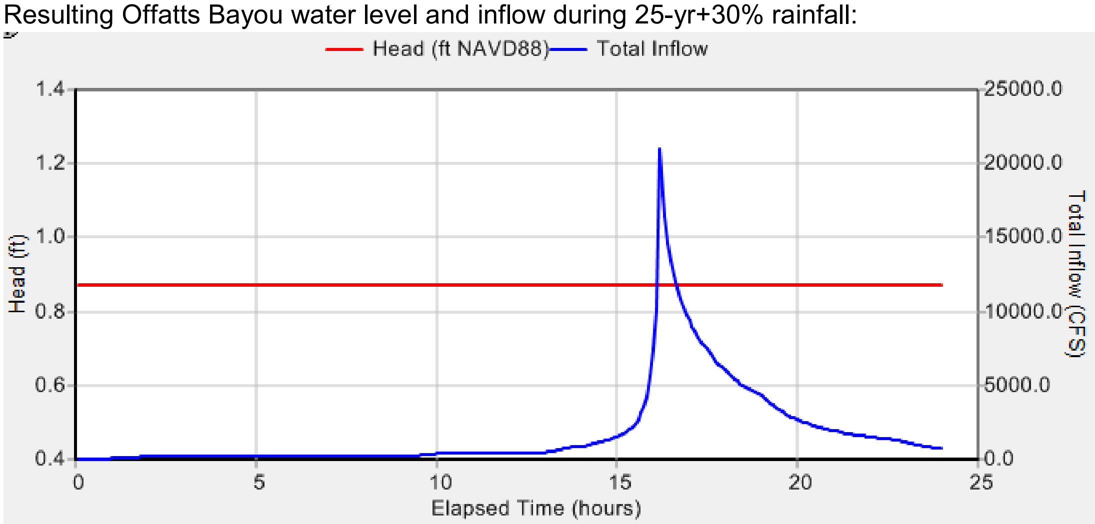
Channel cross section at Offatts Bayou Mouth:



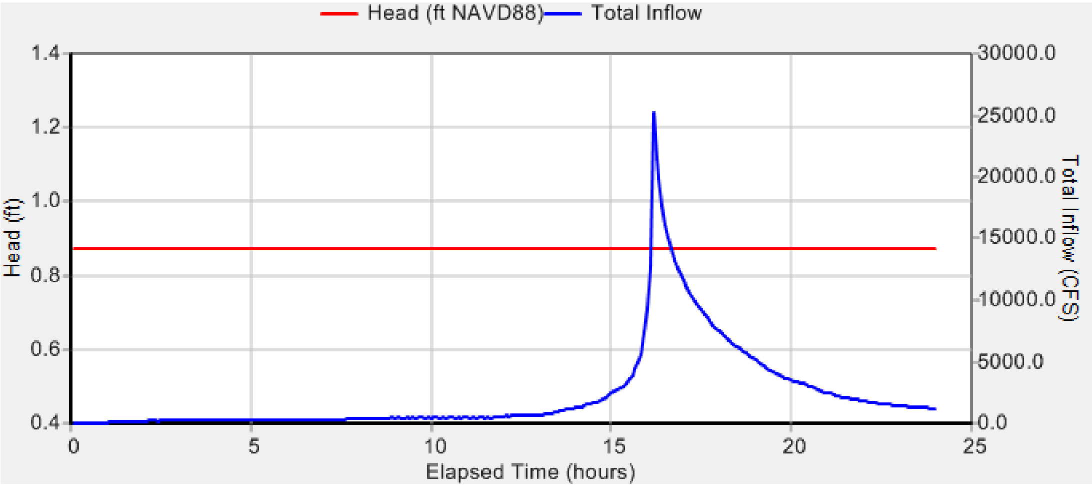
Resulting Offatts Bayou water level and inflow during 10-yr+30% rainfall:



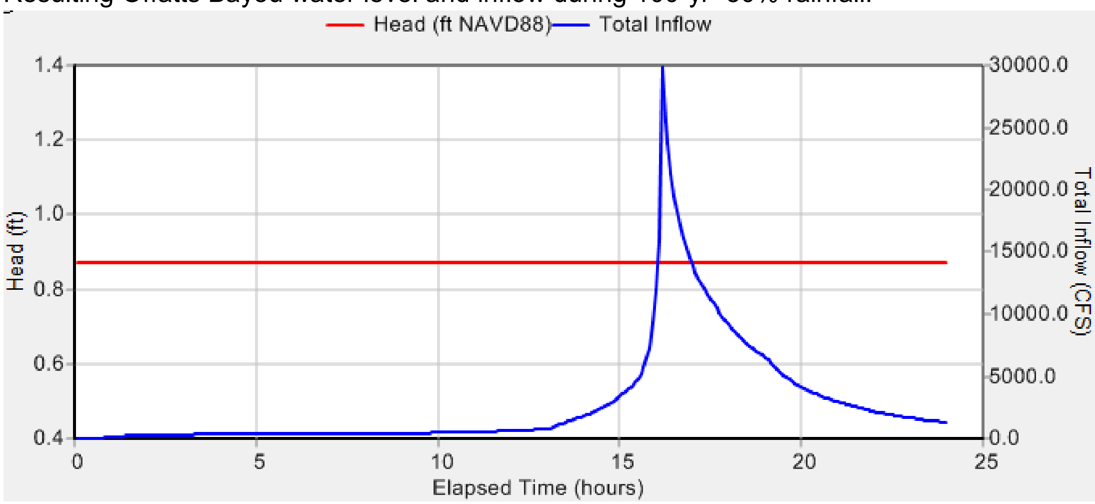
Resulting Offatts Bayou water level and inflow during 25-yr+30% rainfall:



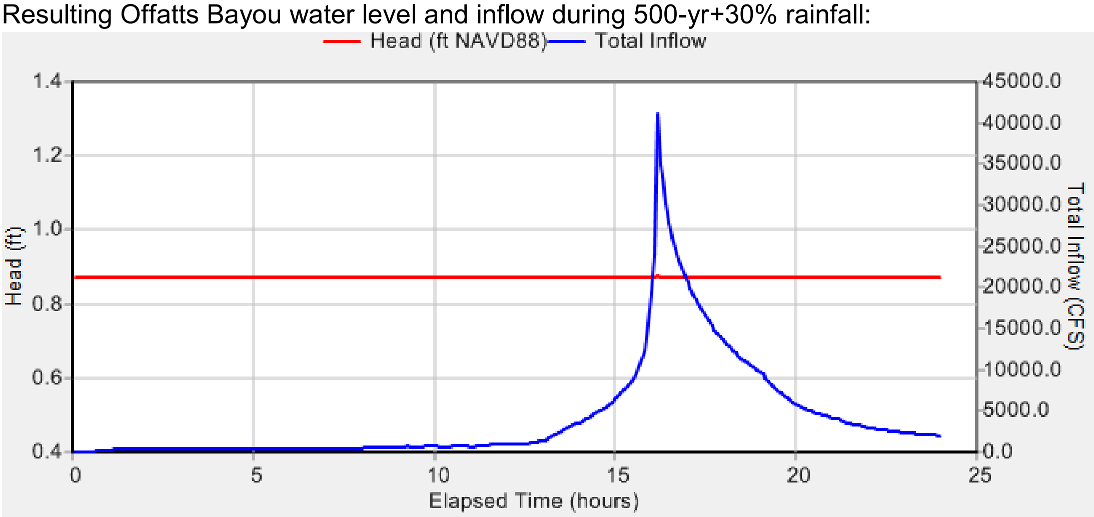
Resulting Offatts Bayou water level and inflow during 50-yr+30% rainfall:



Resulting Offatts Bayou water level and inflow during 100-yr+30% rainfall:

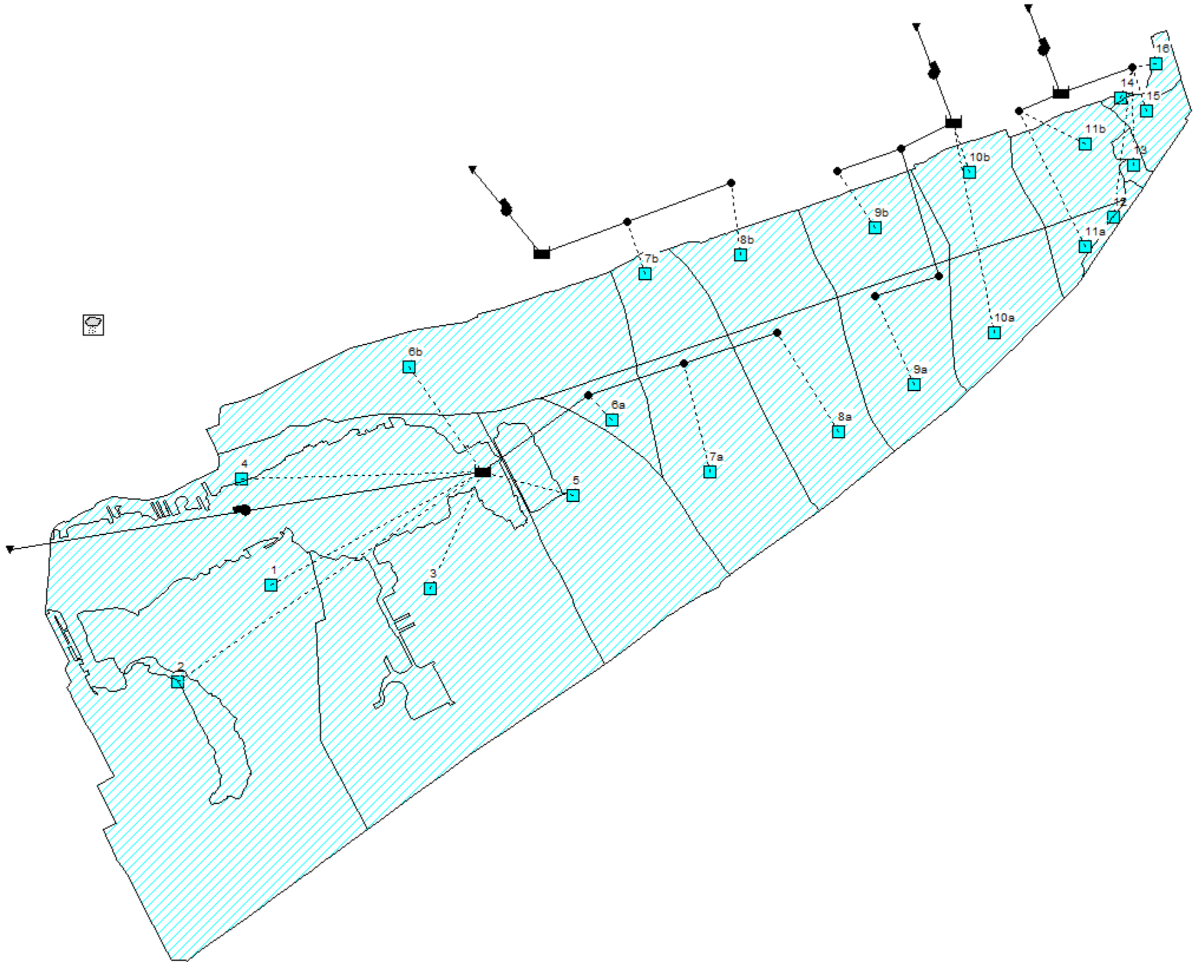


Resulting Offatts Bayou water level and inflow during 500-yr+30% rainfall:

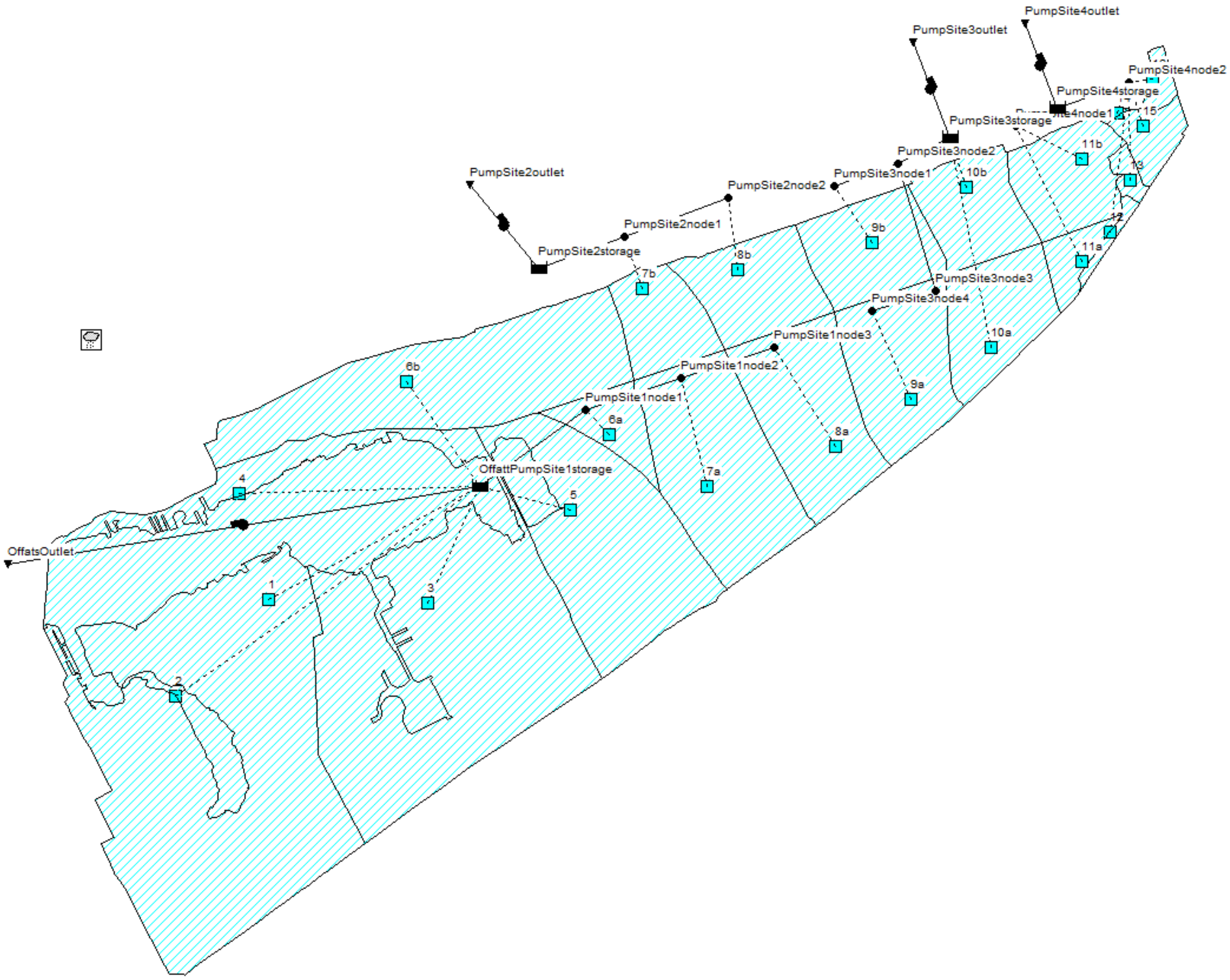


Runs 6: Design Development 25-yr+30% rainfall with gates closed and pumps on

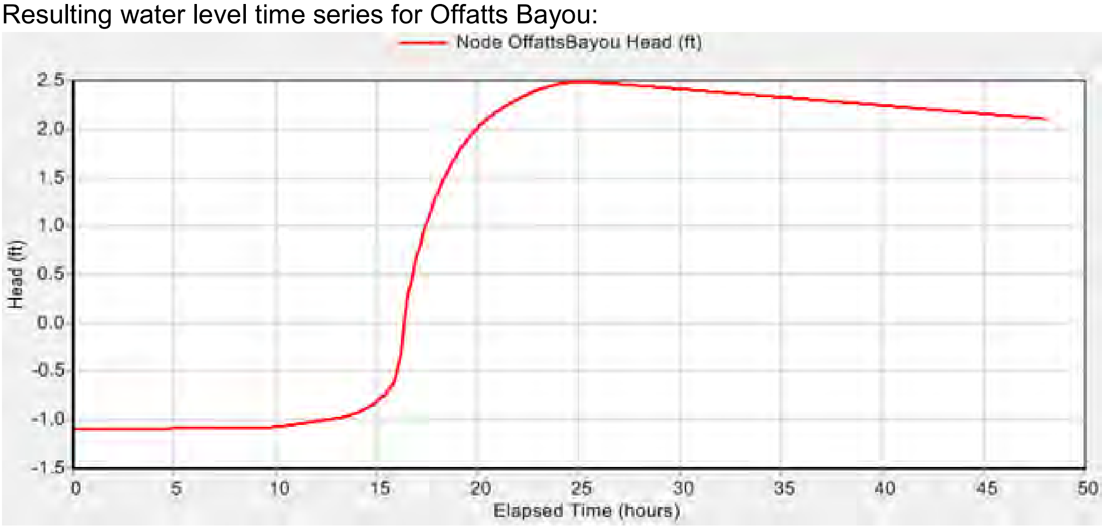
Galveston SWMM model configuration:



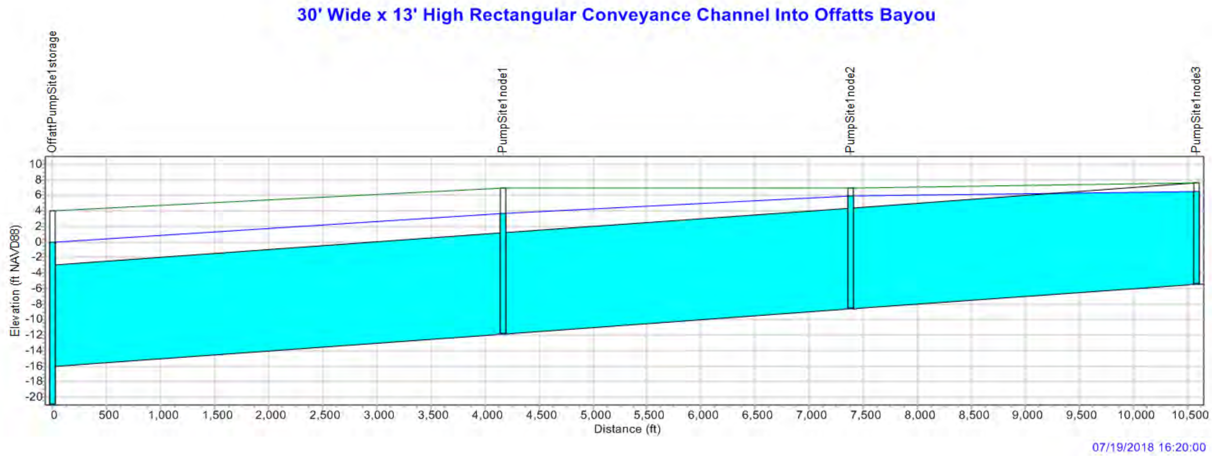
Galveston SWMM model configuration with nodes labeled:

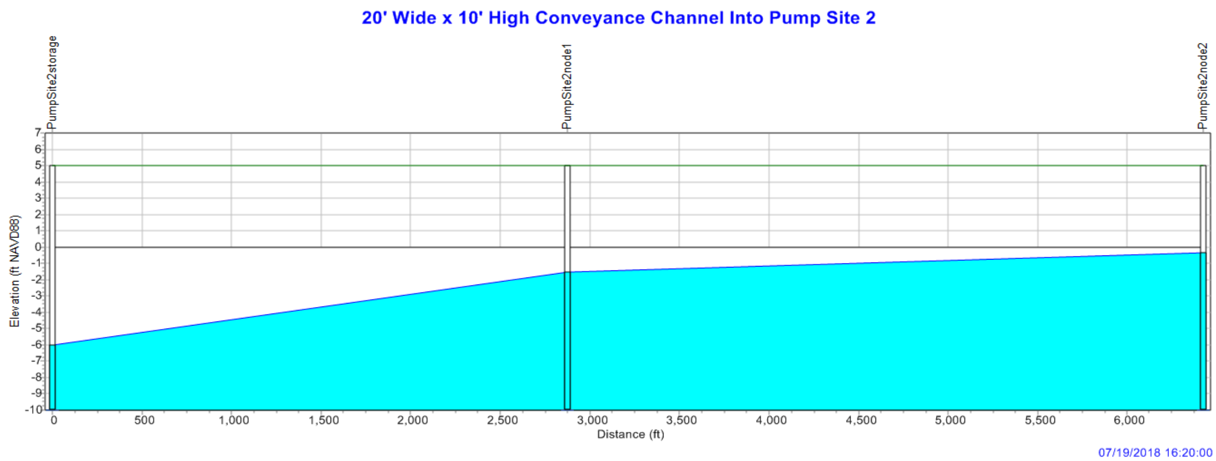


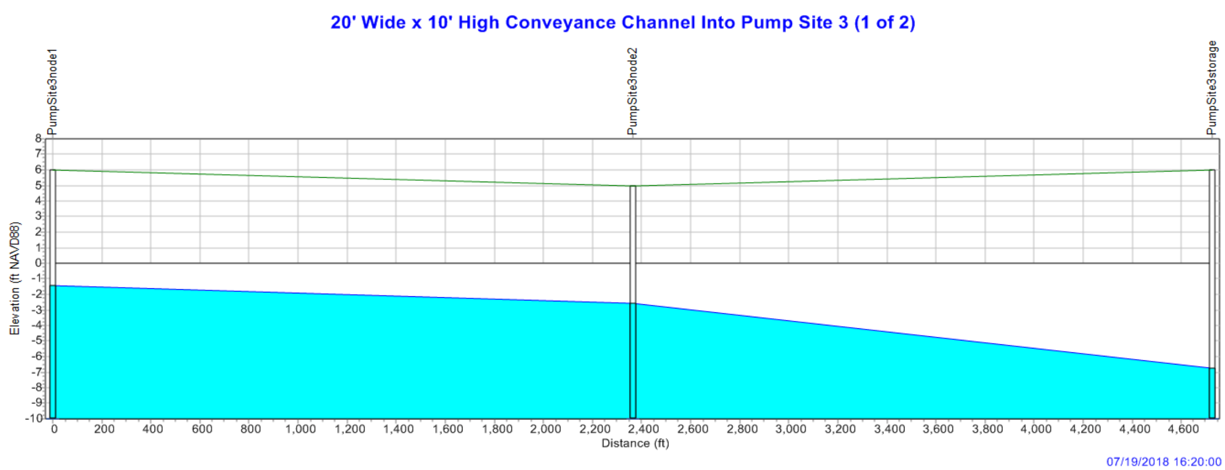
Resulting water level time series for Offatts Bayou:



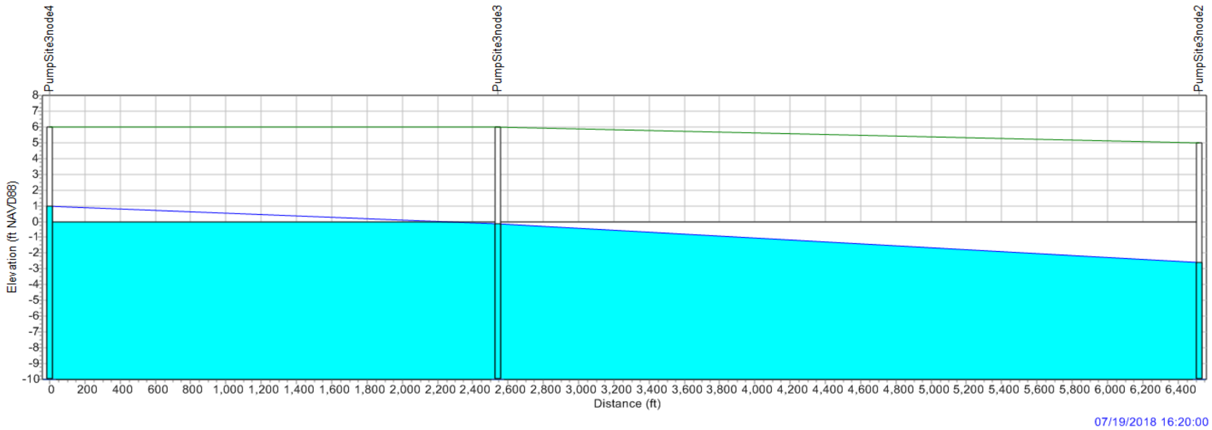
Profiles of peak water level timesteps within the channels:



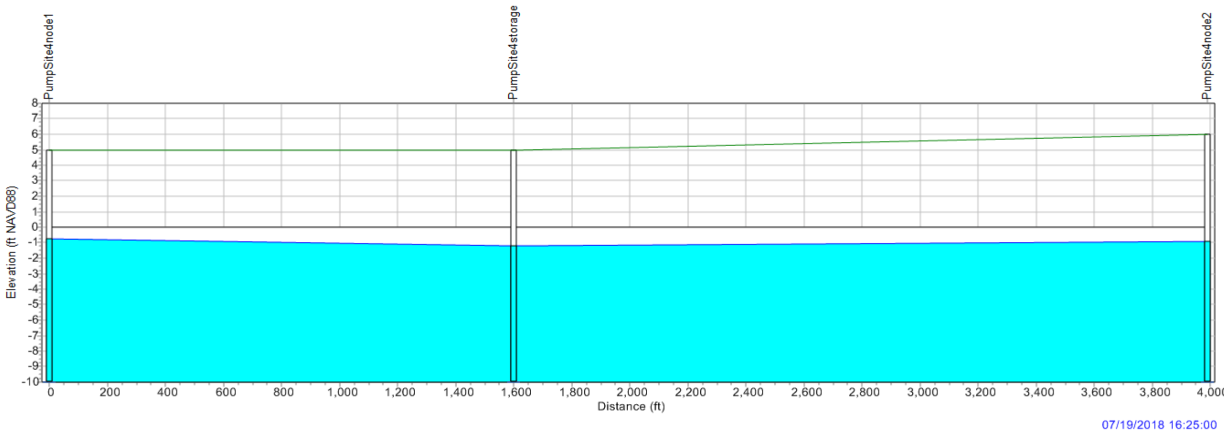




20' Wide x 10' High Conveyance Channel Into Pump Site 3 (2 of 2)



20' Wide x 10' High Conveyance Channel Into Pump Site 4



Runs 7 – 10: Proposed design facilities: gates closed, pumps on

Same model configuration as Run 6.

Resulting flood volumes at each node per rainfall event:

Max Flood Volume (cubic ft) with design 25-yr+30% facilities				
Node	**10-yr+30%**	**50-yr+30%**	**100-yr+30%**	**500-yr+30%**
Offatts Bayou	(131,213,862)*	(33,345,607)*	20,694,898**	157,169,360**
PumpSite1node1	-	117,073	45,792	621,115
PumpSite1node2	-	225,942	454,336	1,134,247
PumpSite1node3	-	-	224,144	621,923
PumpSite2node1	-	-	268,935	634,793
PumpSite2node2	-	22,400	209,863	497,782
PumpSite3node1	-	5,805	136,333	309,439
PumpSite3node2	-	-	179,733	458,969
PumpSite3node3	-	-	-	-
PumpSite3node4	-	-	36,753	125,055
PumpSite4node1	-	21,319	34,605	76,888
PumpSite4node2	-	-	1,844	41,846
Total volume above 4 ft NAVD88:	(131,213,862)*	(32,953,069)*	22,287,234	161,691,417

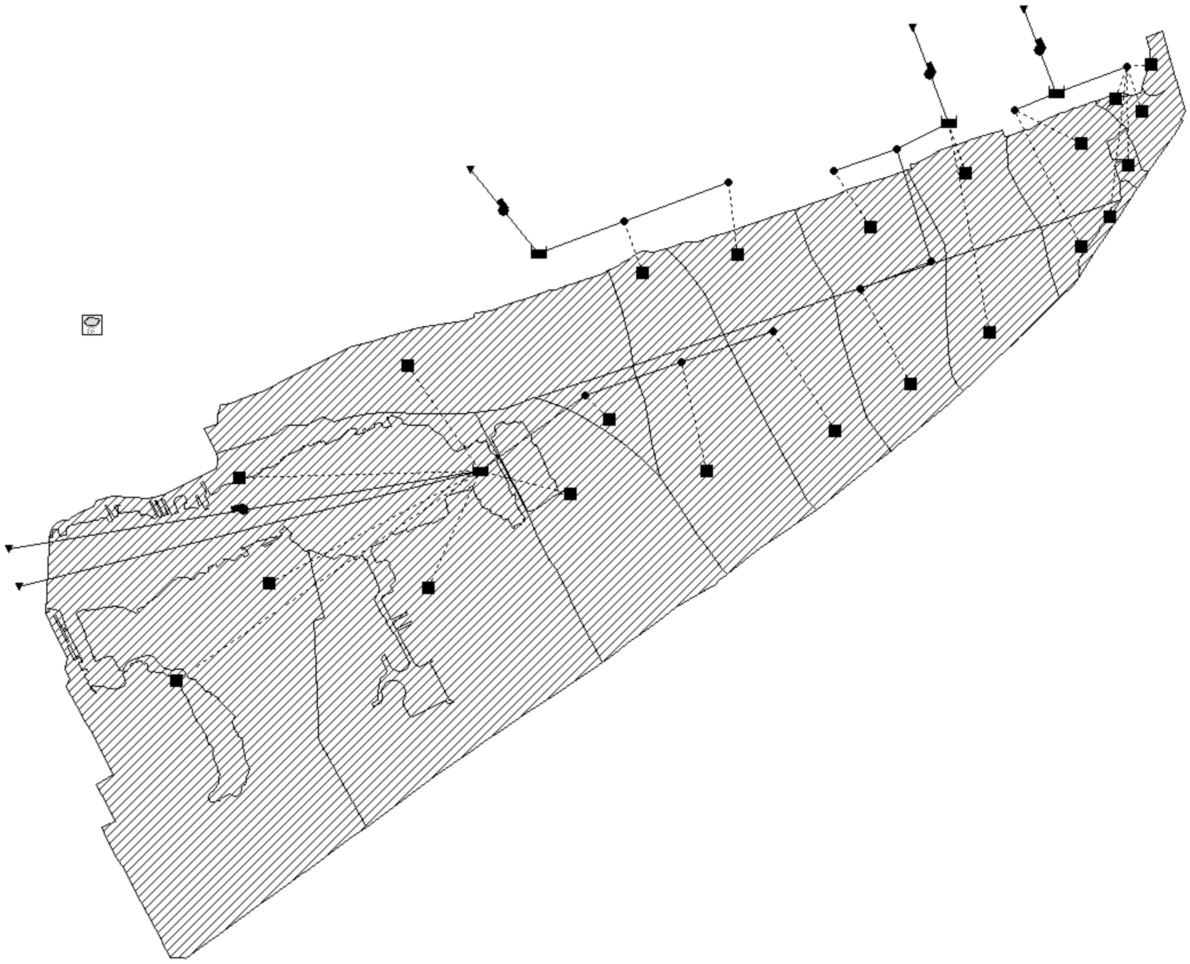
*Values displayed in parenthesis are the volumes of available storage below the 4 ft NAVD88 elevation; i.e., the volume of available storage that still remains in Offatts Bayou.

**Volume of flooding above the 4 ft NAVD88 elevation; i.e., volume of flooding that exceeds the Offatts Bayou storage capacity

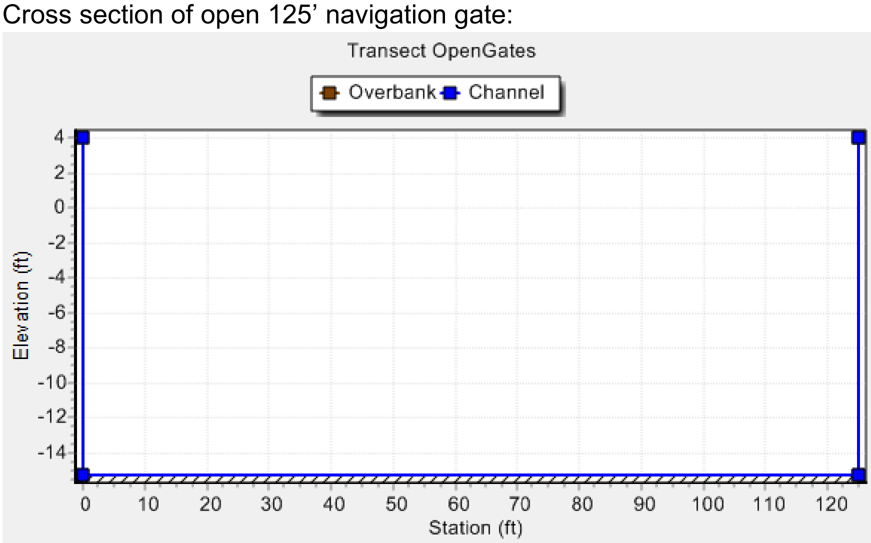
Runs 11 – 15: Proposed design facilities: gates open, pumps on

Only water elevation in Offatts Bayou is evaluated for these runs.

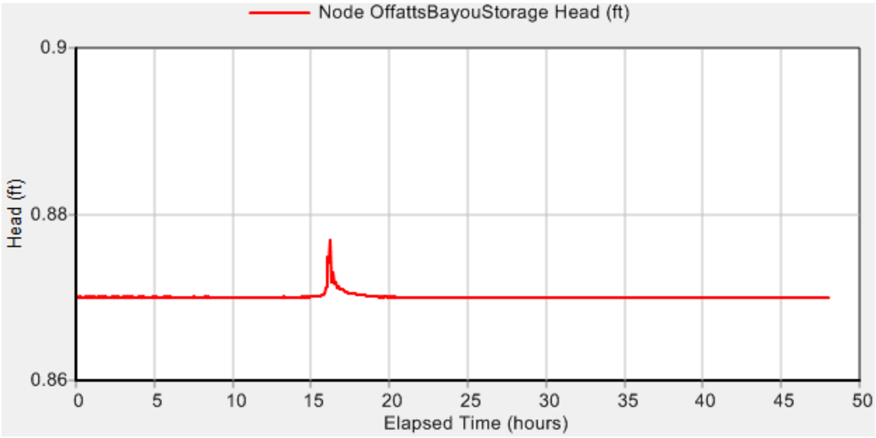
Same model configuration as Run 6, but with simulated open 125' navigation gate:



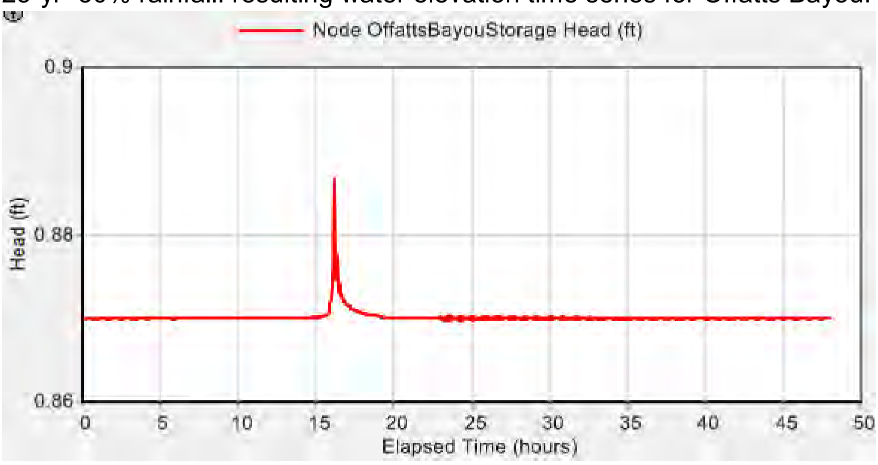
Cross section of open 125' navigation gate:



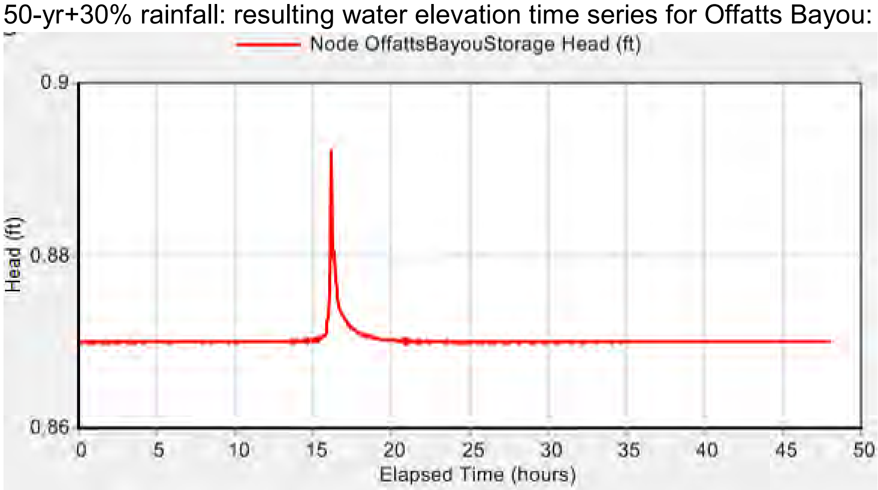
10-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



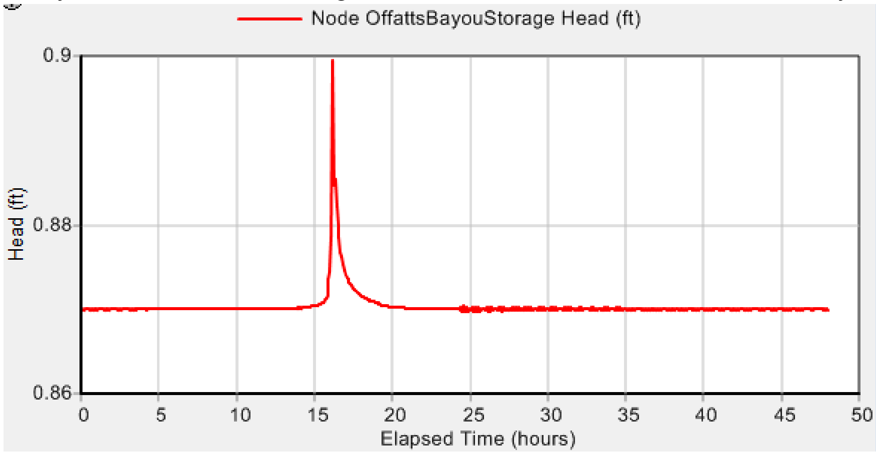
25-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



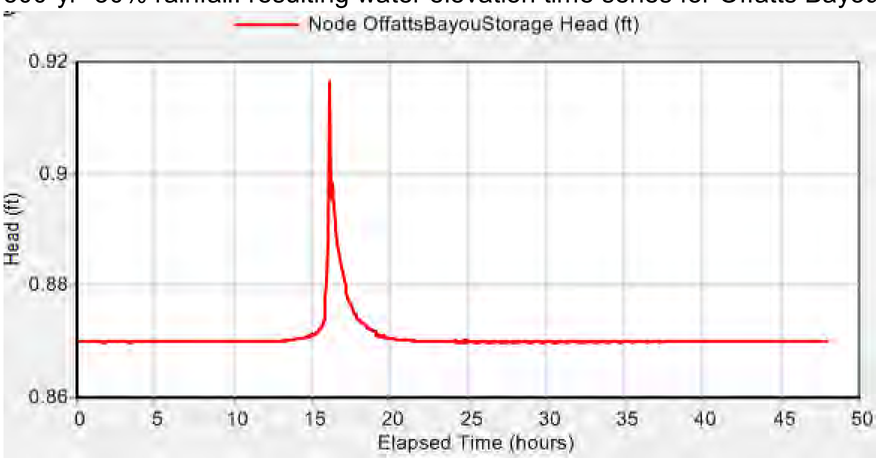
50-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



100-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



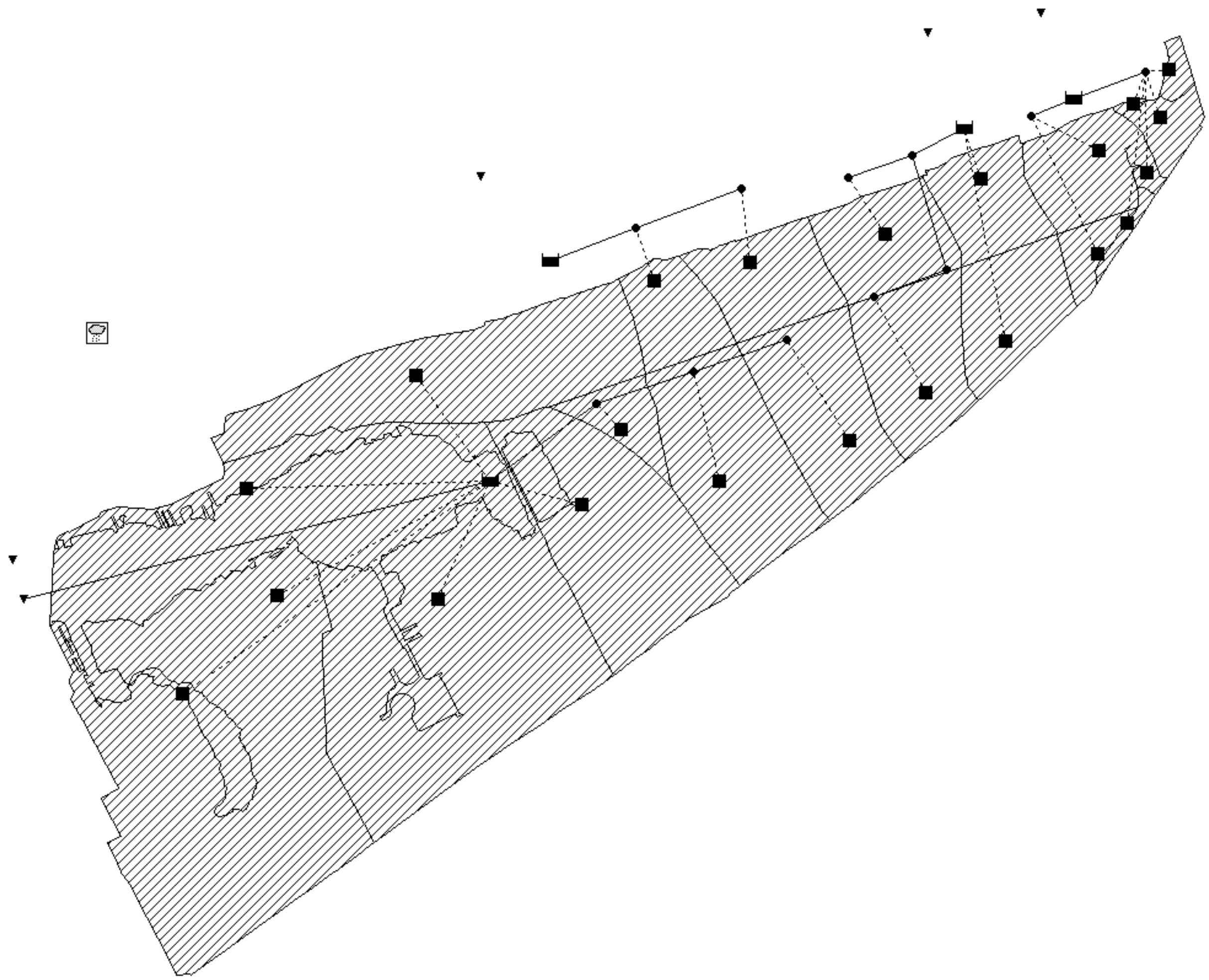
500-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



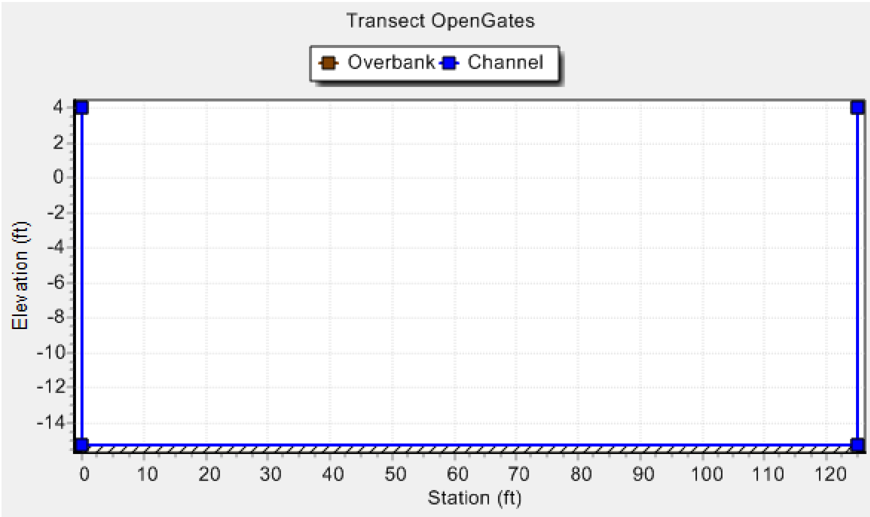
Runs 16 – 20: Proposed design facilities: gates open, pumps off

Only water elevation in Offatts Bayou is evaluated for these runs.

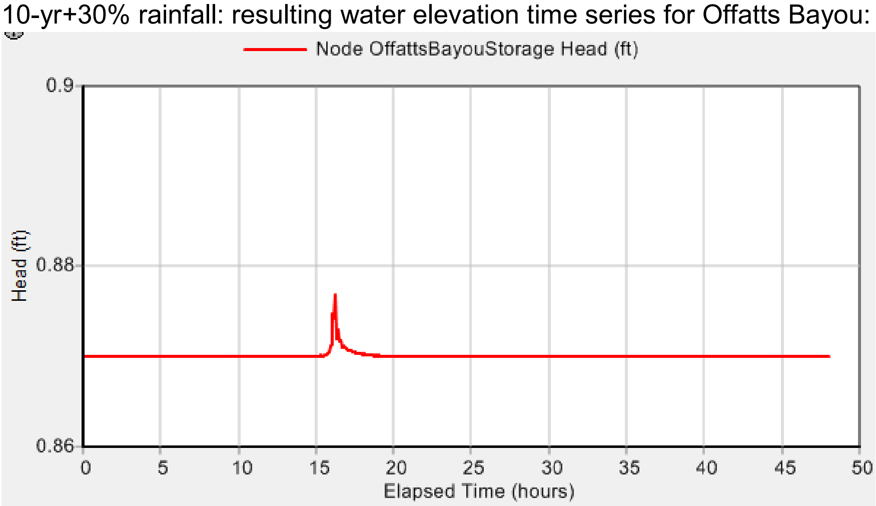
Same model configuration as Run 6, but with simulated open 125’ navigation gate and pumps removed (off):



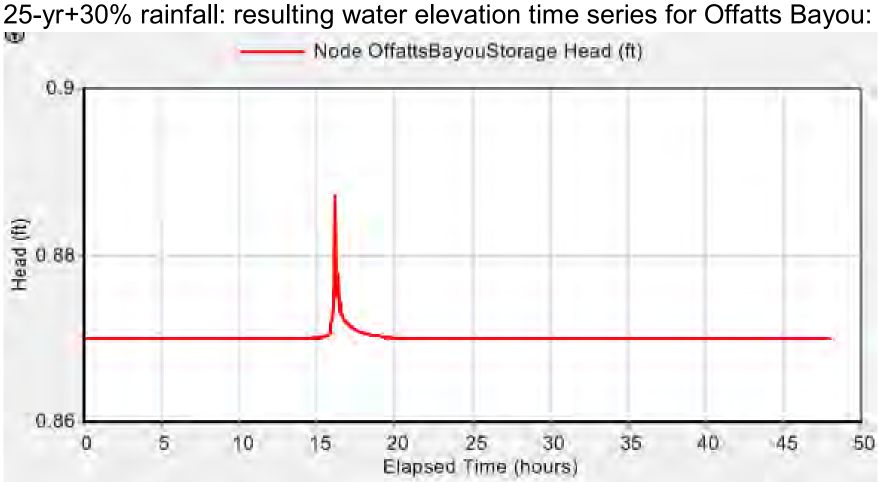
Cross section of open 125' navigation gate:



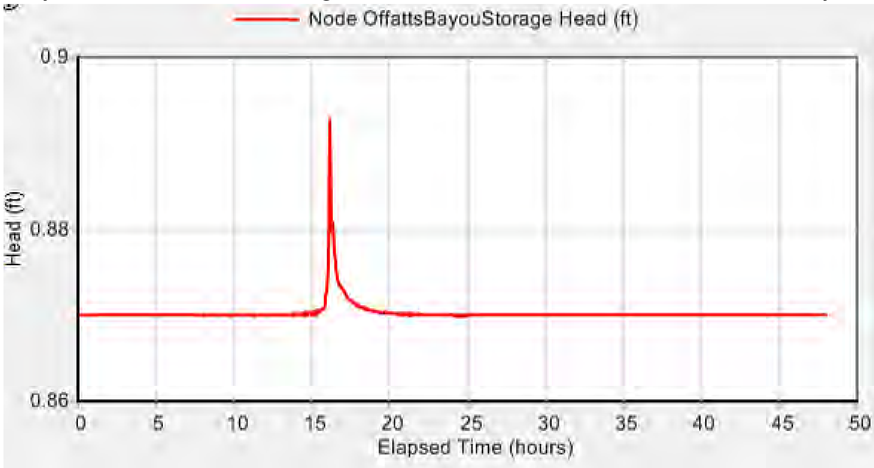
10-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



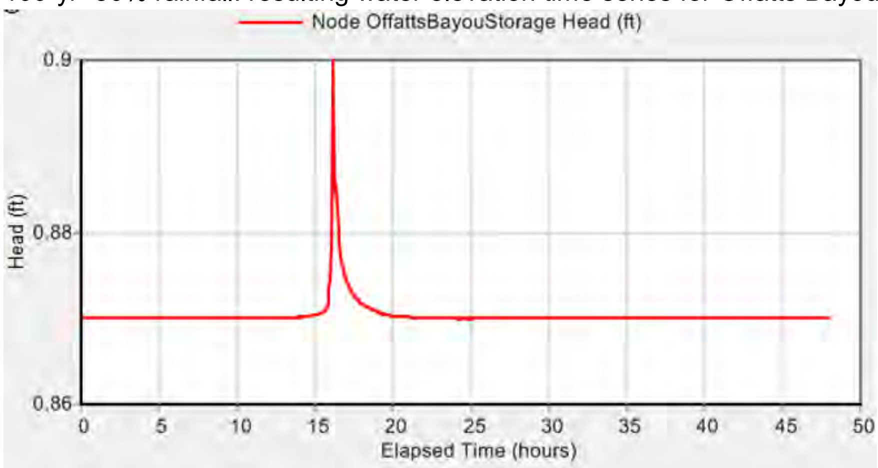
25-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



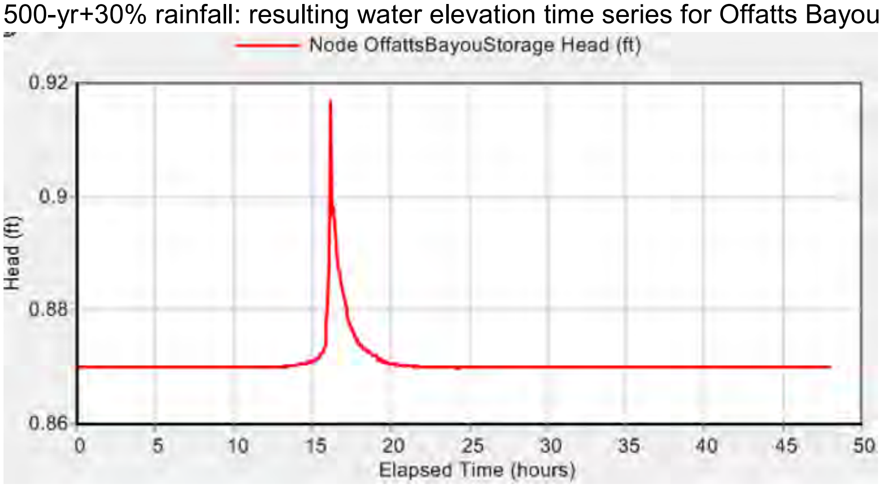
50-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:



100-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:

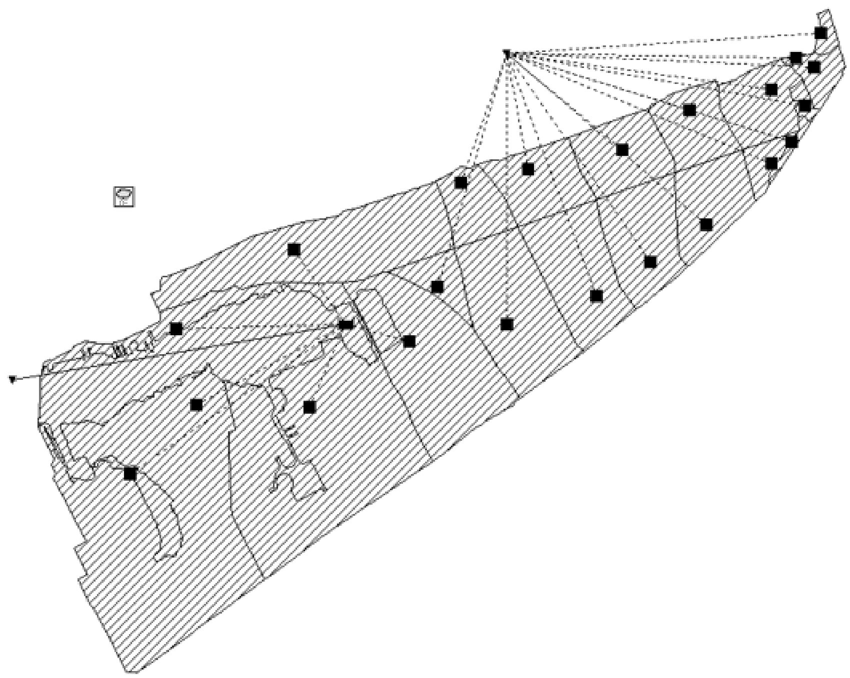


500-yr+30% rainfall: resulting water elevation time series for Offatts Bayou:

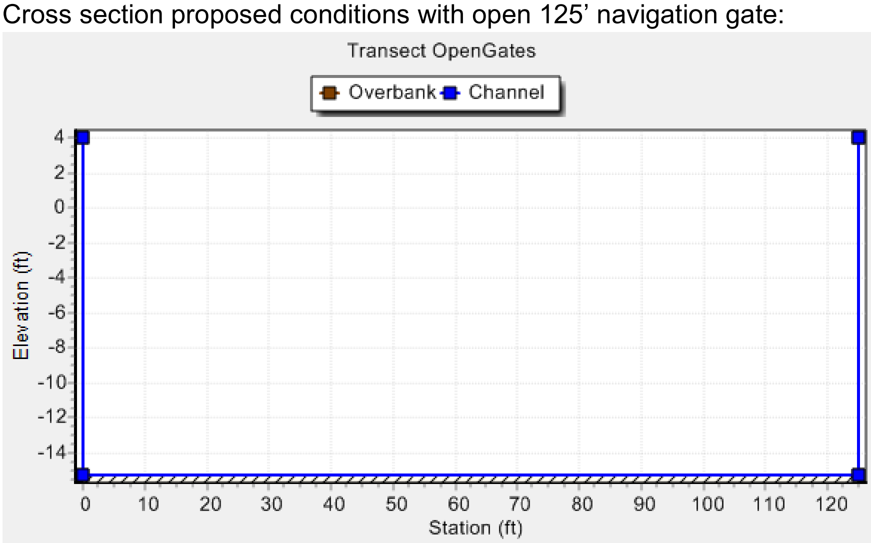


Run 21: Proposed design conditions: gates open, tidal run

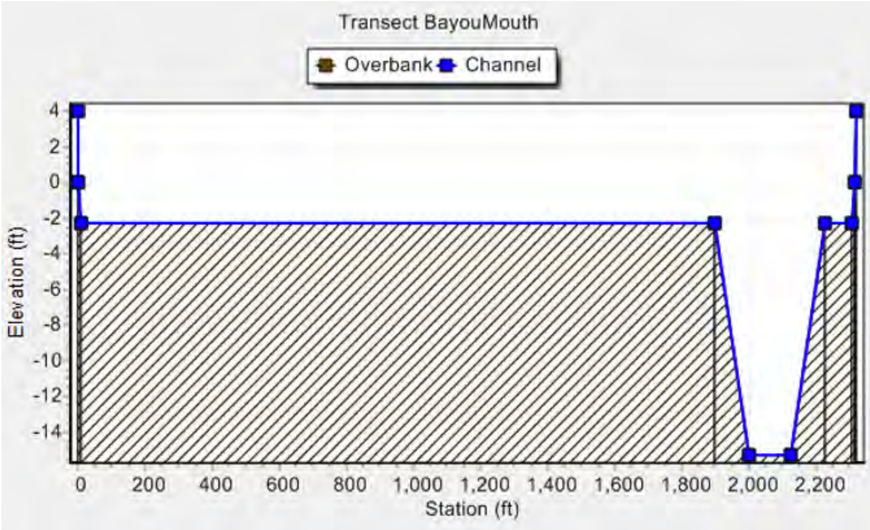
SWMM model configures set up only to evaluate water level in Offatts Bayou with 3-day tidal boundary condition at the mouth of the Bayou:



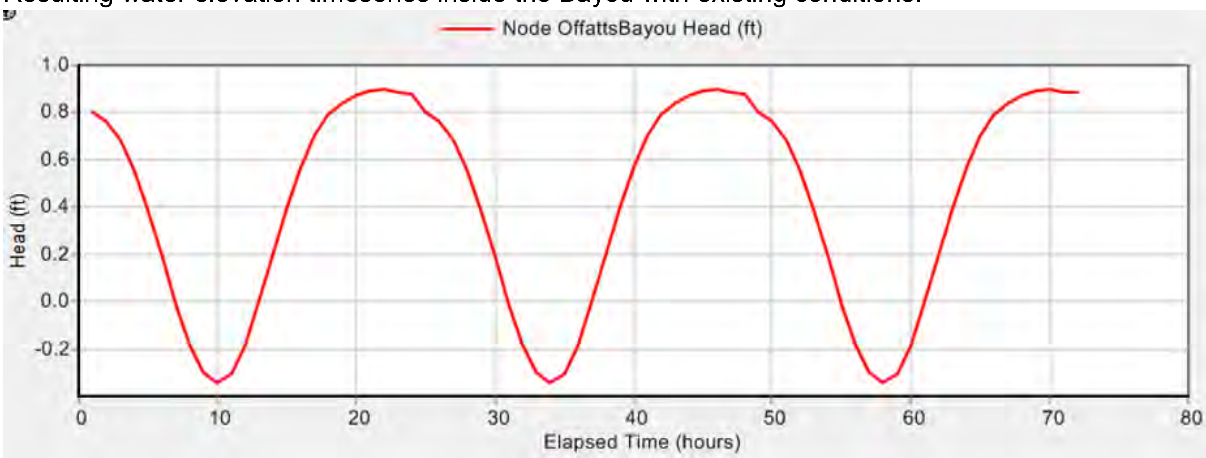
Cross section proposed conditions with open 125' navigation gate:



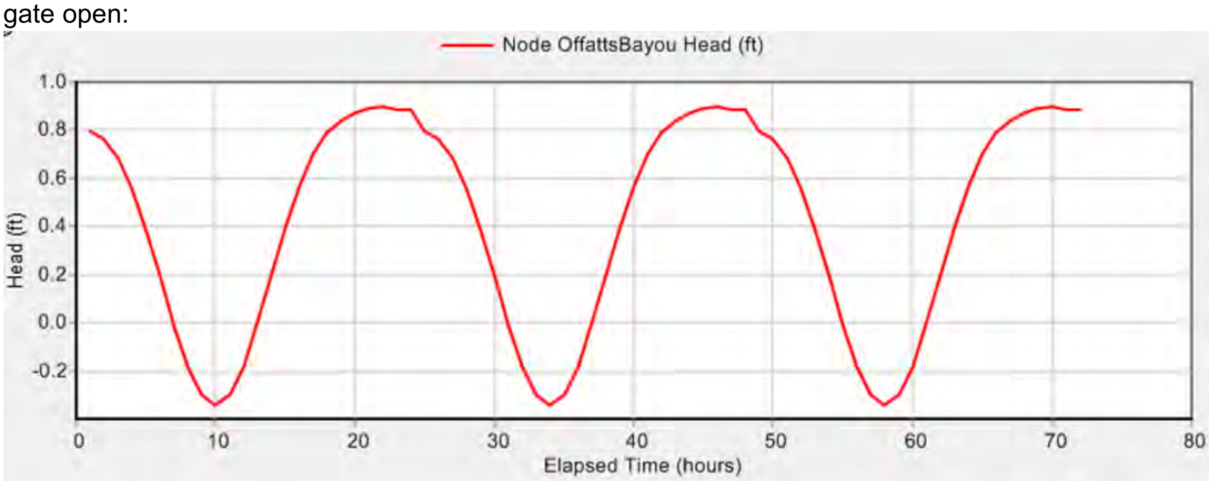
Cross section of the existing conditions at the mouth of the Bayou:



Resulting water elevation timeseries inside the Bayou with existing conditions:



Resulting water elevation timeseries inside the Bayou with proposed floodwall and navigation gate open:



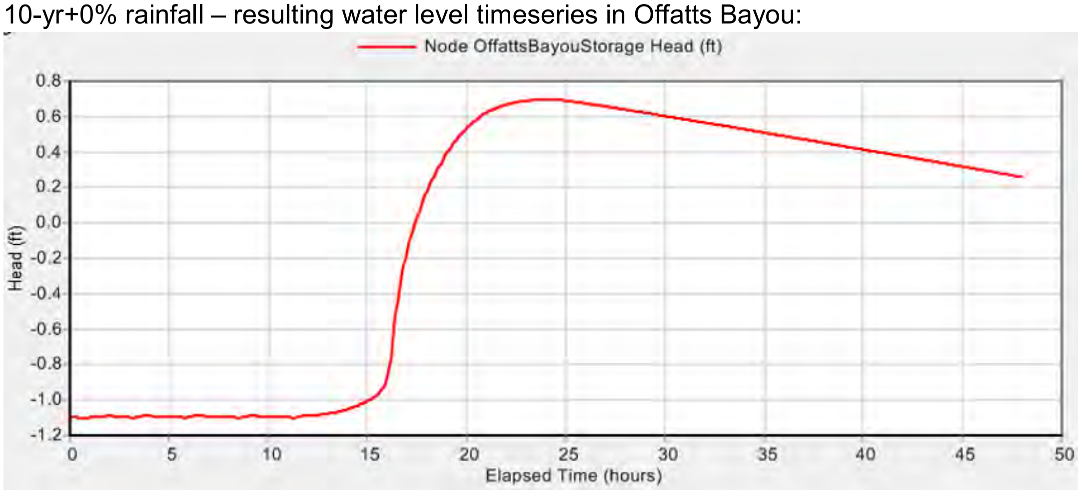
Run 23 – 36: Alternative facility design development with gates closed, pumps on

Same model layout as Run 6, but with the following sized facilities:

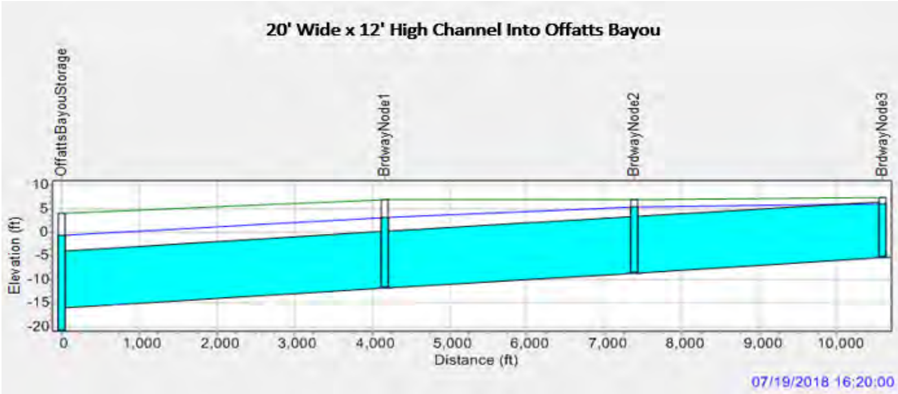
Run	Rainfall Event	Offatts Pump Site 1 Size (cfs)	Pump Site 2 Size (cfs)	Pump Site 3 Size (cfs)	Pump Site 4 Size (cfs)	Offatts Pump Site 1: Channel Dimensions (ft) and Cross Section Area (sq ft)		Pump Site 2: Channel Dimensions (ft) and Cross Section Area (sq ft)		Pump Site 3: Channel Dimensions (ft) and Cross Section Area (sq ft)		Pump Site 4: Channel Dimensions (ft) and Cross Section Area (sq ft)		Peak Water Elevation. In Offatts Bayou (ft NAVD88)
23	10-yr+0%	250	1,500	4,500	1,500	20' wide x 12' high	240	15' wide x 10' high	150	15' wide x 8' high	120	10' wide x 8' high	80	0.7
24	10-yr+15%	250	1,500	4,500	1,500	25' wide x 12' high	300	15' wide x 10' high	150	15' wide x 9' high	135	10' wide x 9' high	90	1.1
25	10-yr+30%	250	1,500	4,500	1,500	28' wide x 12' high	336	16' wide x 10' high	160	15' wide x 10' high	150	10' wide x 10' high	100	1.5
26	25-yr+0%	250	1,500	4,500	1,500	26' wide x 12' high	312	15' wide x 10' high	150	15' wide x 10' high	150	10' wide x 10' high	100	1.4
27	25-yr+15%	250	1,500	4,500	1,500	30' wide x 12' high	360	15' wide x 11' high	165	15' wide x 11' high	165	12' wide x 10' high	120	1.95
28	50-yr+0%	250	1,500	4,500	1,500	29' wide x 12' high	348	15' wide x 11' high	165	15' wide x 11' high	165	11' wide x 10' high	110	2.05
29	50-yr+15%	250	1,500	4,500	1,500	38' wide x 12' high	456	15' wide x 12' high	180	15' wide x 12' high	180	15' wide x 12' high	180	2.7
30	50-yr+30%	2,000	1,500	4,500	1,700	40' wide x 12' high	480	20' wide x 12' high	240	20' wide x 12' high	240	20' wide x 12' high	240	2
31	100-yr+0%	250	1,500	4,500	1,500	38' wide x 12' high	456	15' wide x 12' high	180	15' wide x 12' high	180	15' wide x 11' high	165	2.8
32	100-yr+15%	2,000	1,500	4,500	1,700	40' wide x 12' high	480	20' wide x 12' high	240	20' wide x 12' high	240	20' wide x 12' high	240	2.2
33	100-yr+30%	4,000	2,000	5,000	2,100	48' wide x 12' high	576	20' wide x 12' high	240	20' wide x 12' high	240	20' wide x 12' high	240	1.95
34	500-yr+0%	4,000	2,000	6,000	2,100	48' wide x 12' high	576	20' wide x 12' high	240	20' wide x 12' high	240	20' wide x 12' high	240	2.5
35	500-yr+15%	6,000	2,500	6,100	2,500	54' wide x 12' high	648	22' wide x 12' high	264	20' wide x 14' high	280	20' wide x 12' high	240	2.4
36	500-yr+30%	10,000	3,300	7,500	3,300	60' wide x 12' high	720	25' wide x 12' high	300	30' wide x 14' high	420	20' wide x 12' high	240	2

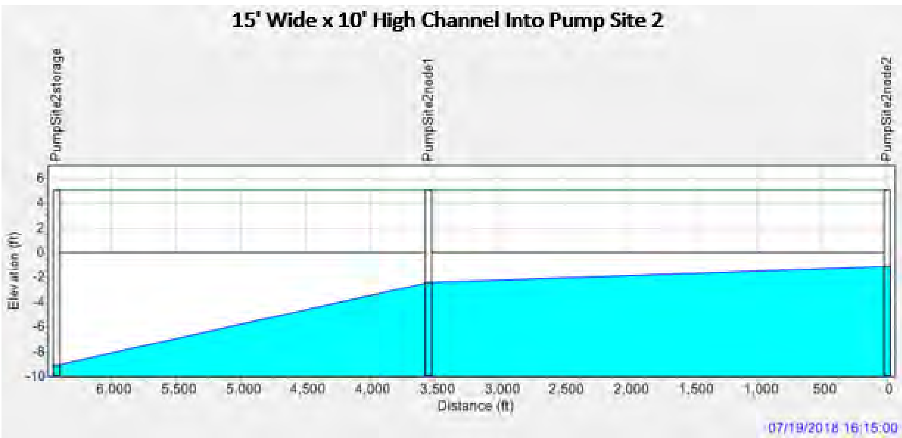
All run results assume no overtopping along the proposed flood barrier.

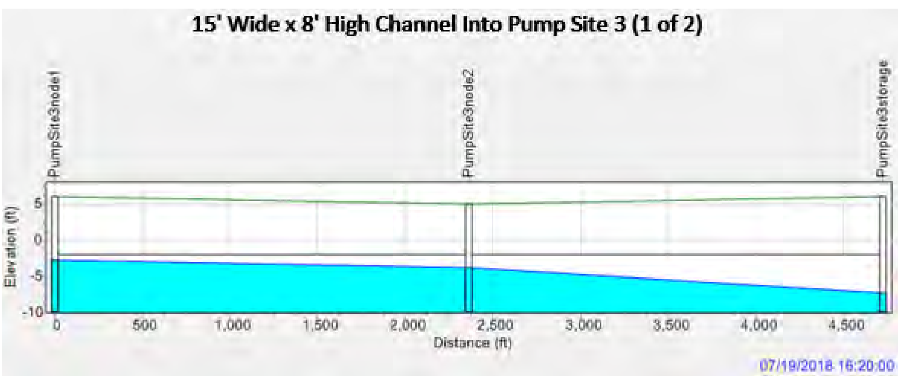
10-yr+0% rainfall – resulting water level timeseries in Offatts Bayou:

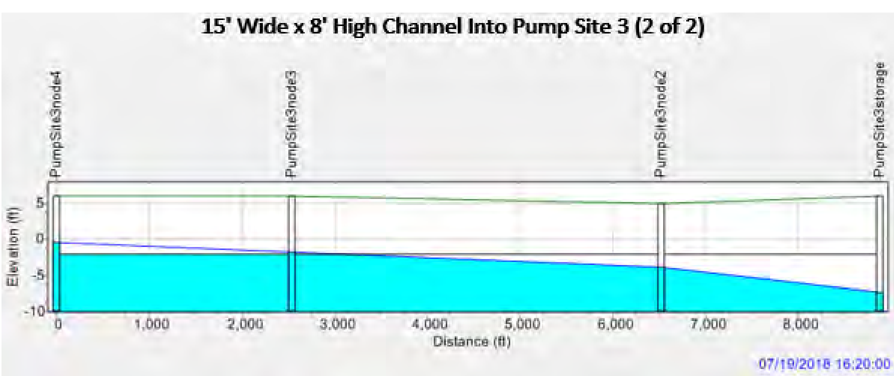


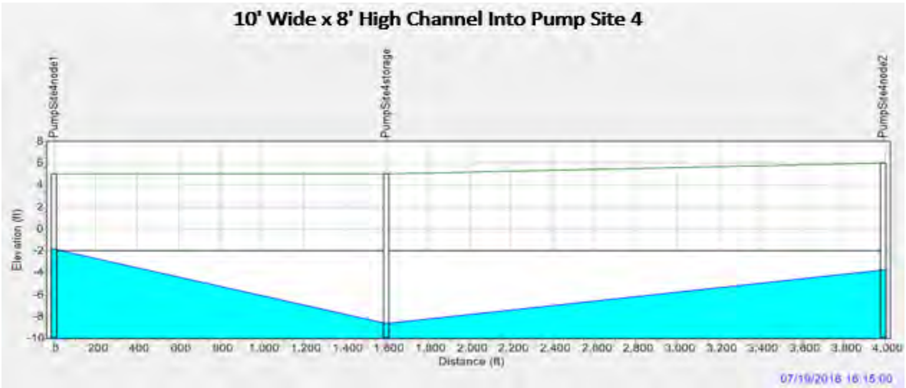
10-yr+0% rainfall – Profiles of peak water level timesteps within the channels:



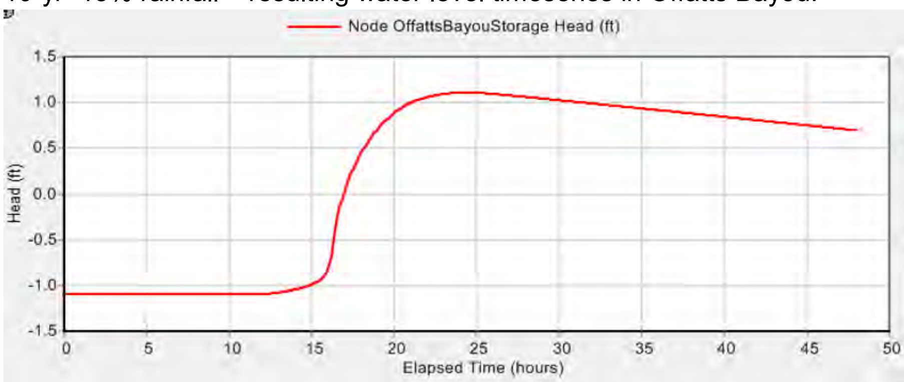




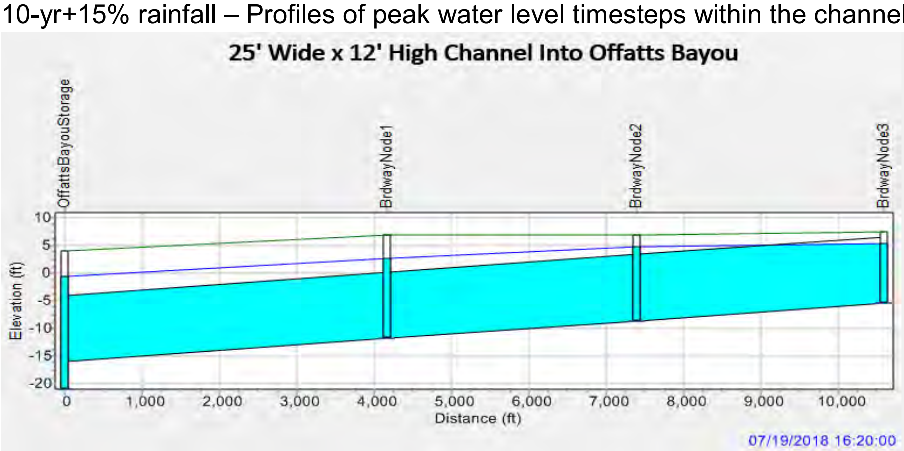


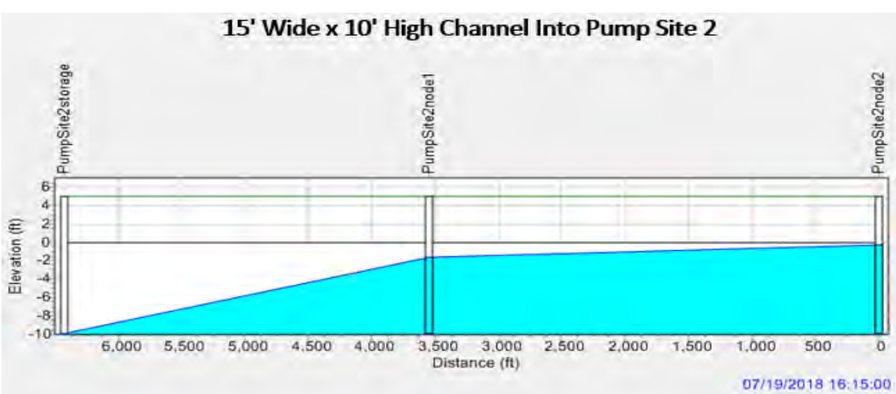


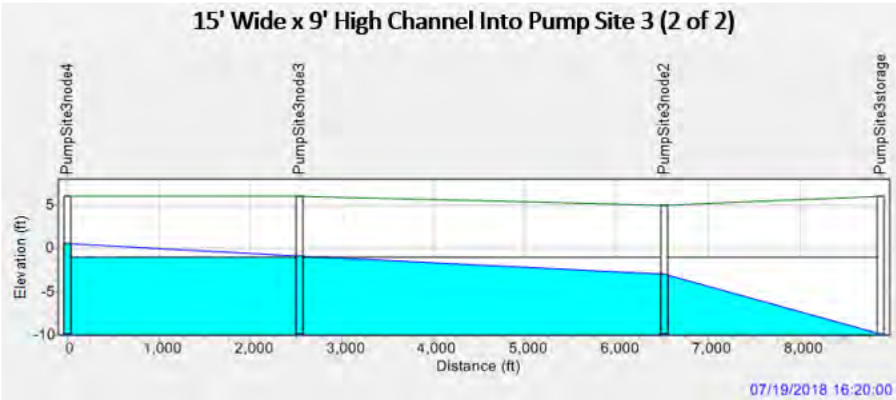
10-yr+15% rainfall – resulting water level timeseries in Offatts Bayou:

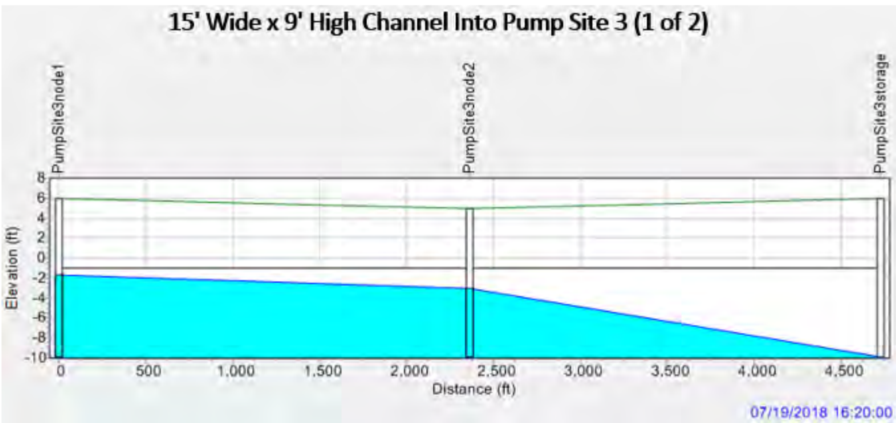


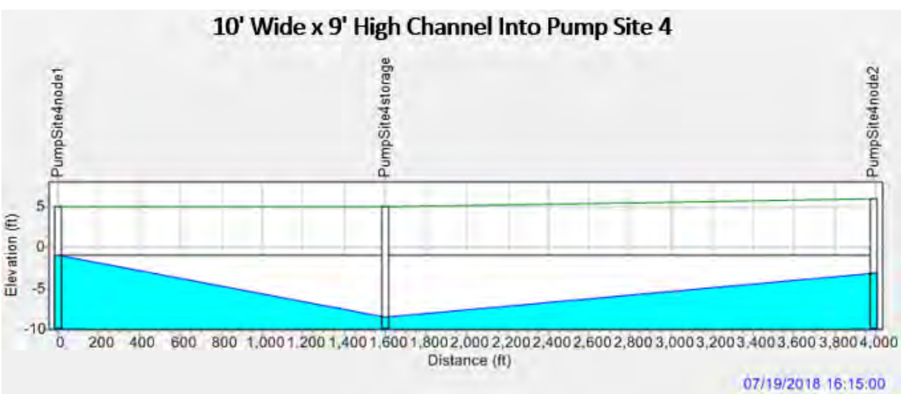
10-yr+15% rainfall – Profiles of peak water level timesteps within the channels:



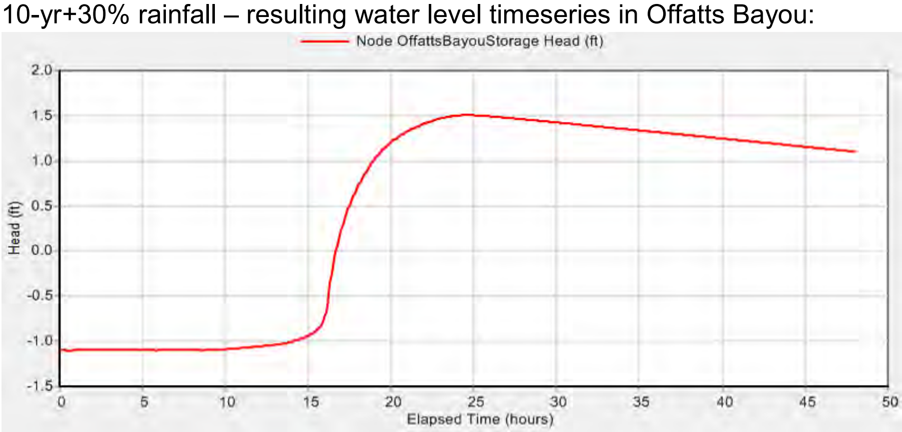




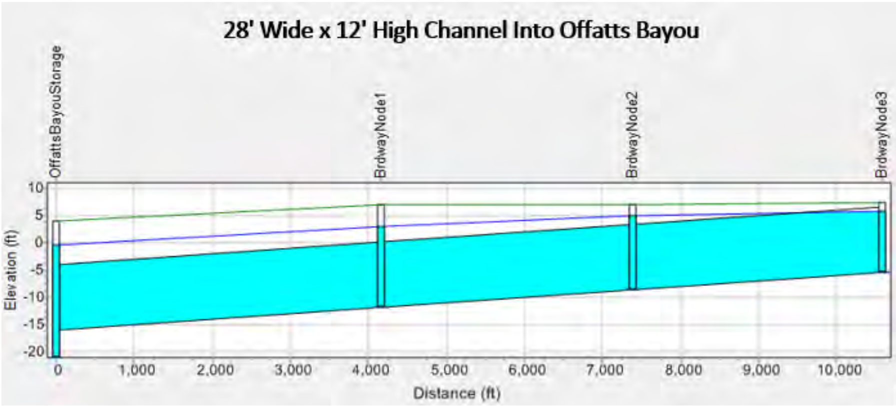


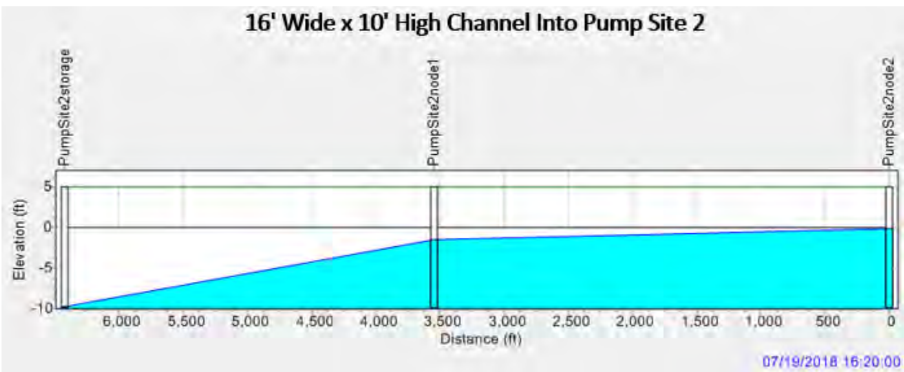


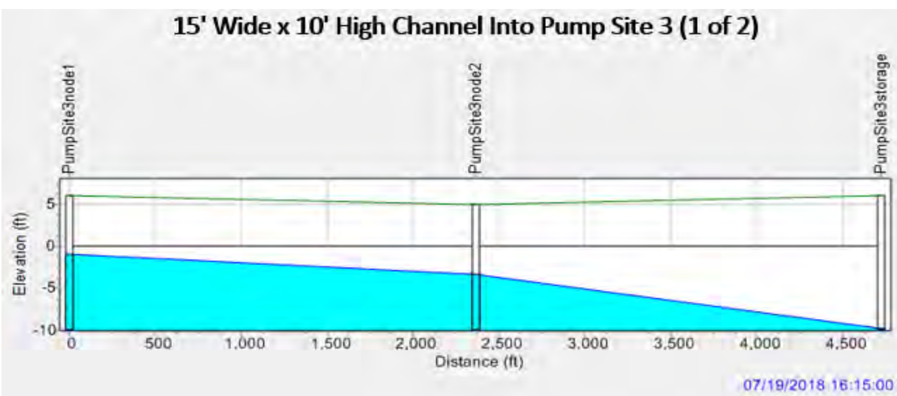
10-yr+30% rainfall – resulting water level timeseries in Offatts Bayou:

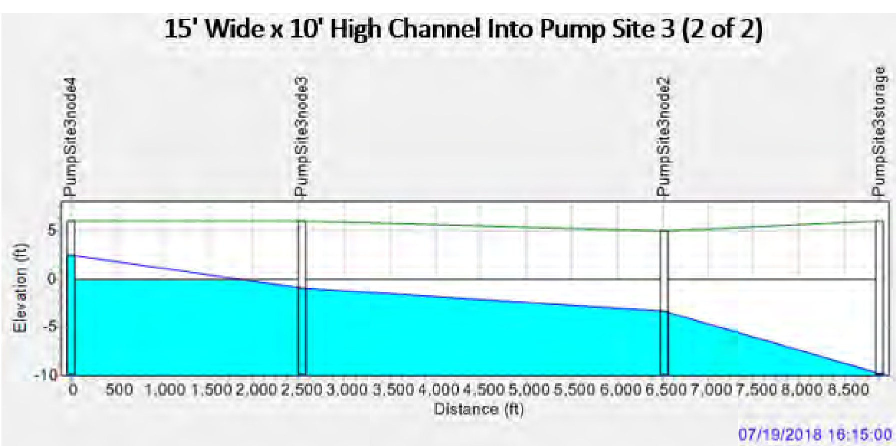


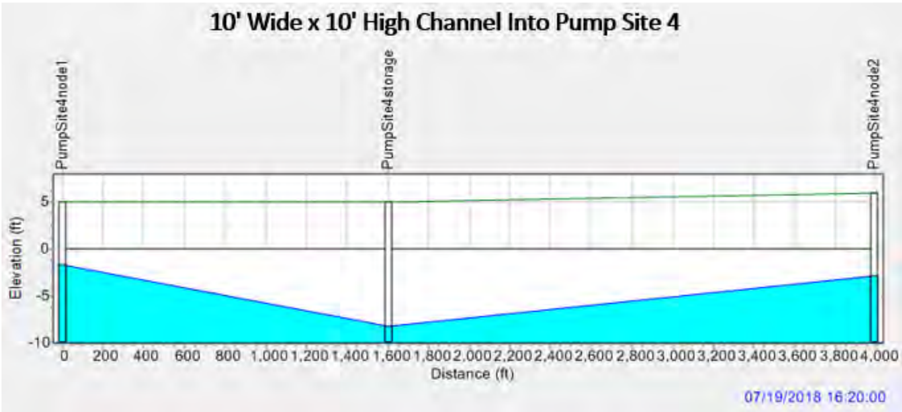
10-yr+30% rainfall – Profiles of peak water level timesteps within the channels:



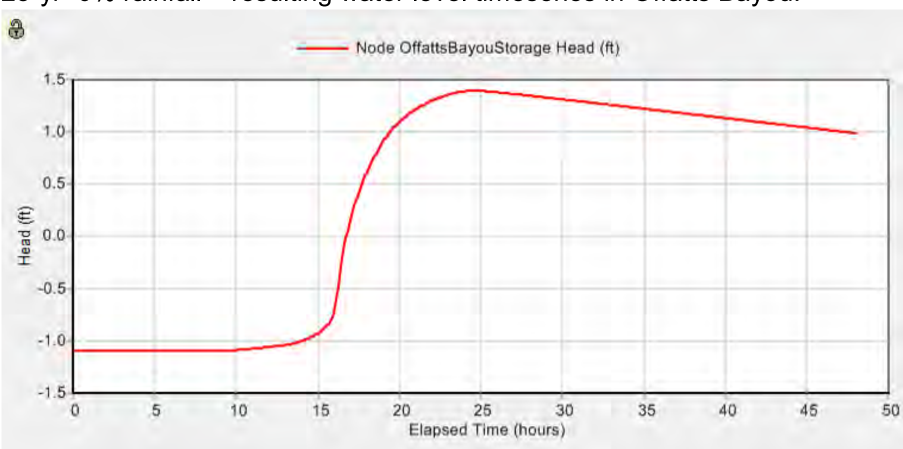




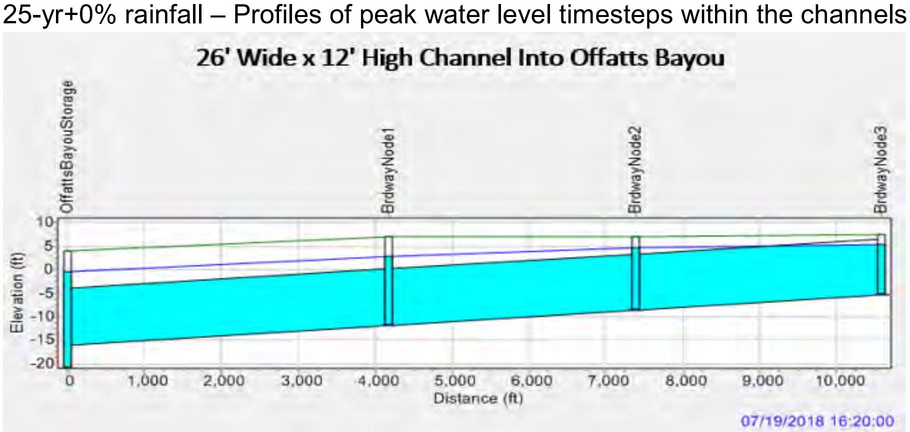


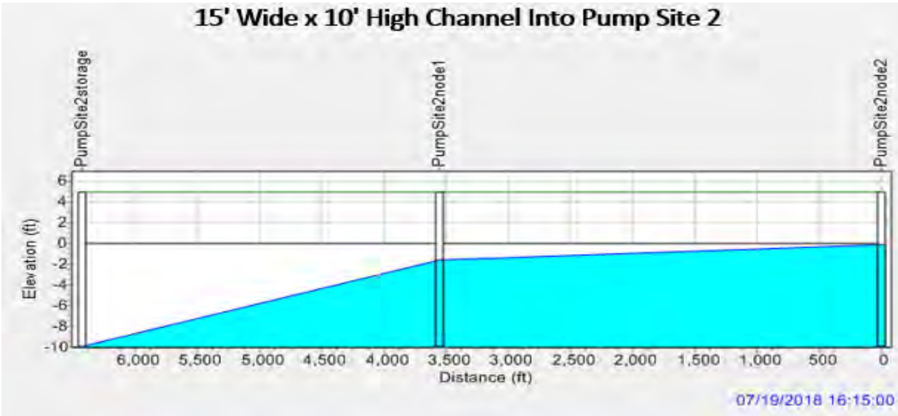


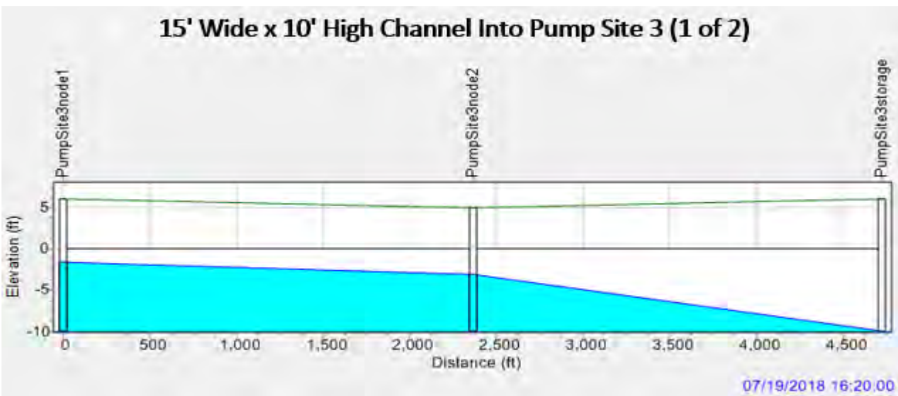
25-yr+0% rainfall – resulting water level timeseries in Offatts Bayou:

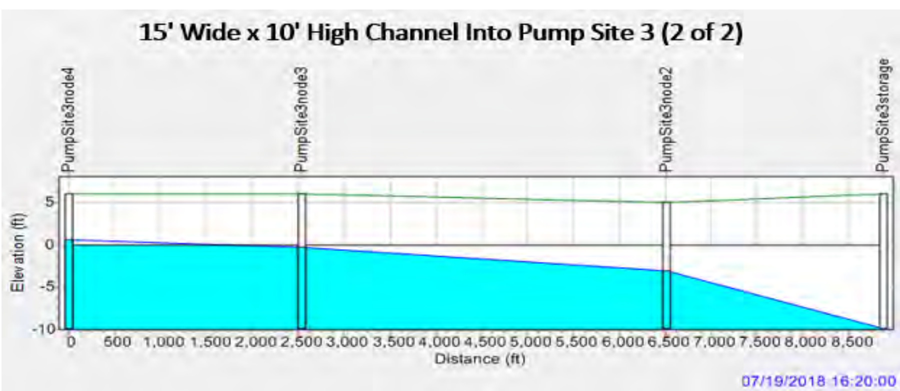


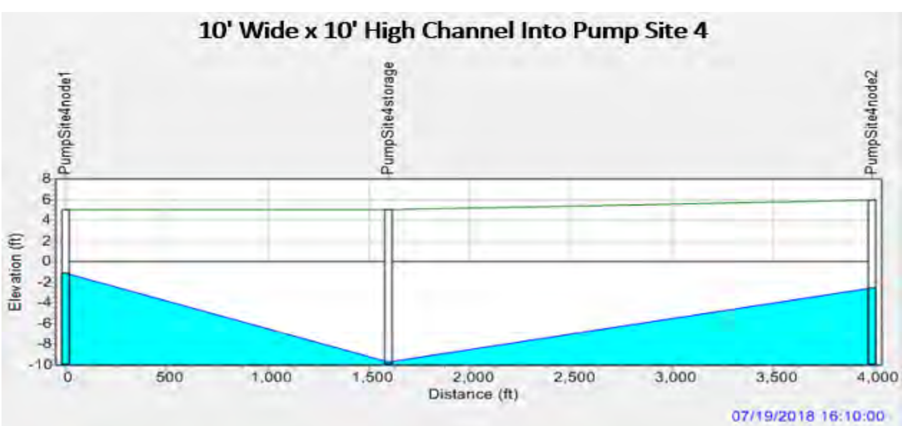
25-yr+0% rainfall – Profiles of peak water level timesteps within the channels:



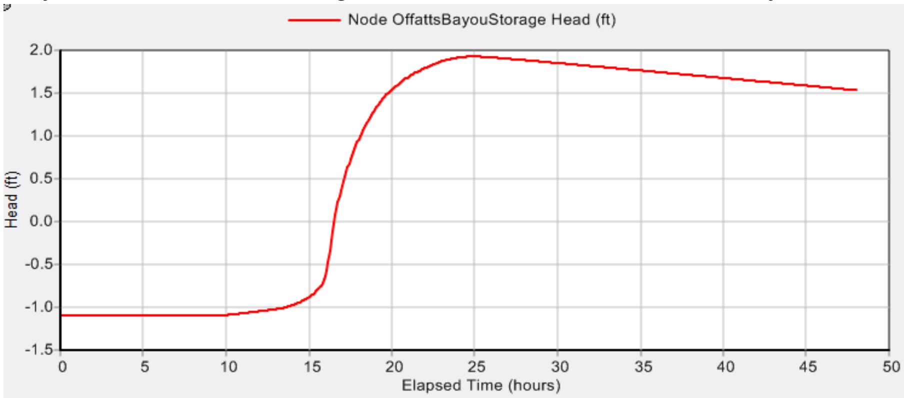

15' Wide x 10' High Channel Into Pump Site 2
PumpSite2storage
PumpSite2node1
PumpSite2node2
Elevation (ft)
Distance (ft)
07/19/2018 16:15:00


15' Wide x 10' High Channel Into Pump Site 3 (1 of 2)
PumpSite3node1
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:20:00

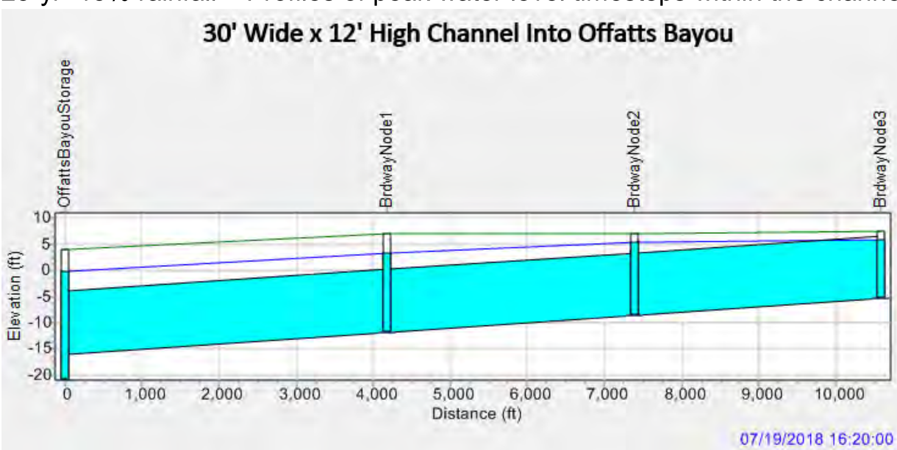

15' Wide x 10' High Channel Into Pump Site 3 (2 of 2)
PumpSite3node4
PumpSite3node3
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:20:00

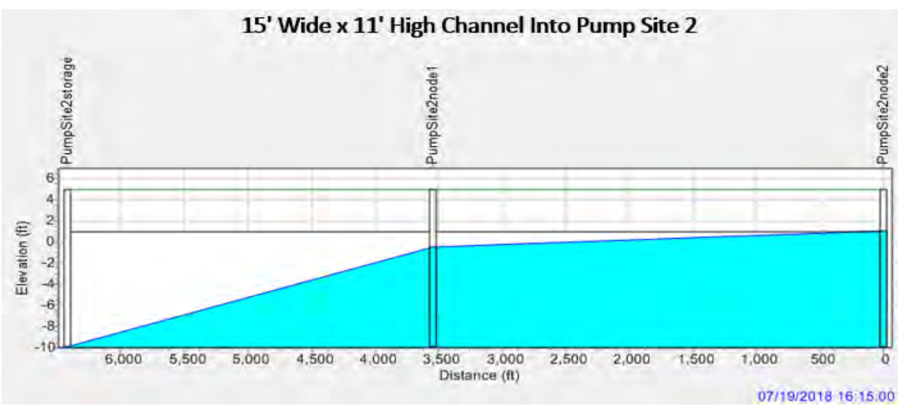

10' Wide x 10' High Channel Into Pump Site 4
PumpSite4node1
PumpSite4storage
PumpSite4node2
Elevation (ft)
Distance (ft)
07/19/2018 16:10:00

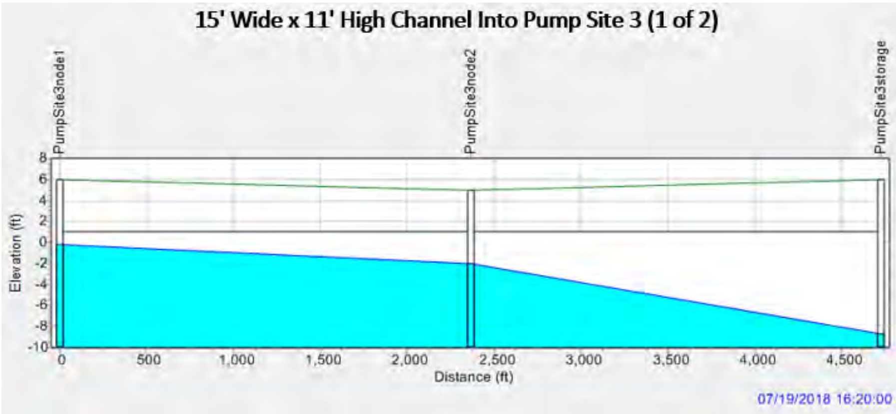
25-yr+15% rainfall – resulting water level timeseries in Offatts Bayou:

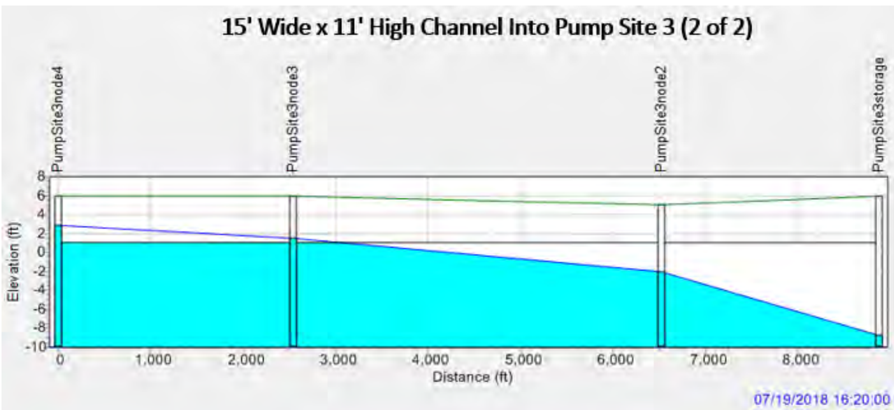


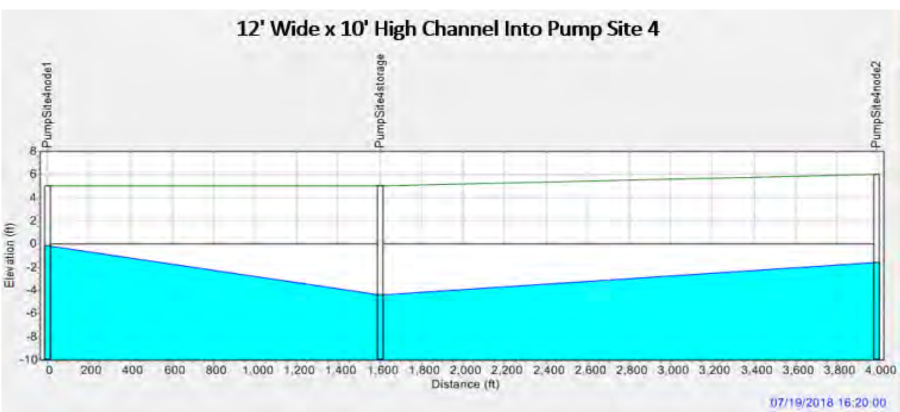
25-yr+15% rainfall – Profiles of peak water level timesteps within the channels:



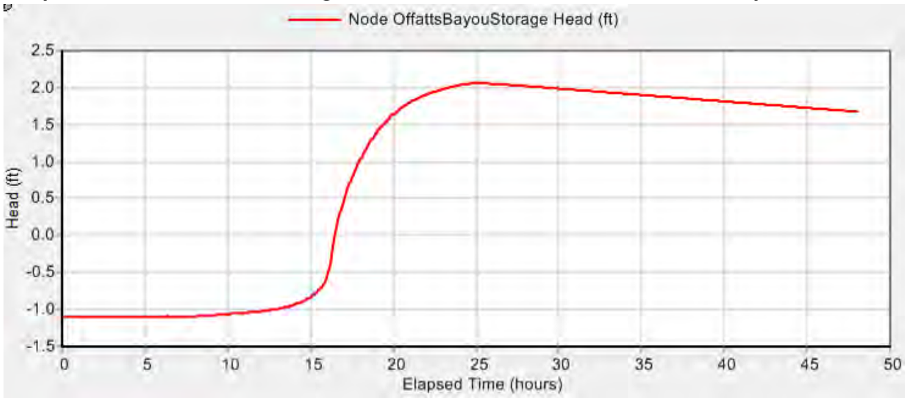



15' Wide x 11' High Channel Into Pump Site 3 (1 of 2)
PumpSite3node1
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:20:00

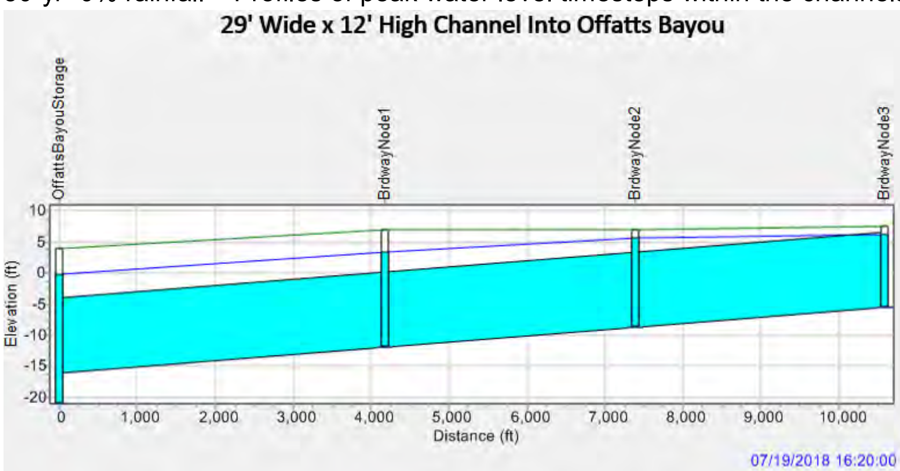

15' Wide x 11' High Channel Into Pump Site 3 (2 of 2)
PumpSite3node4
PumpSite3node3
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:20:00

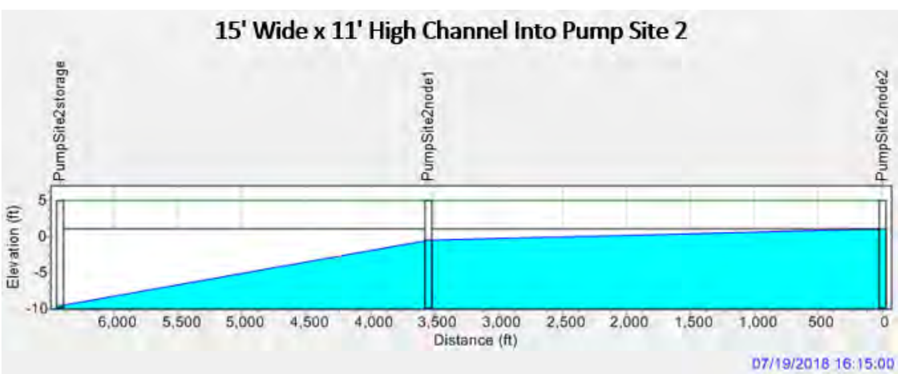

12' Wide x 10' High Channel Into Pump Site 4
PumpSite4node1
PumpSite4storage
PumpSite4node2
Elevation (ft)
Distance (ft)
07/19/2018 16:20:00

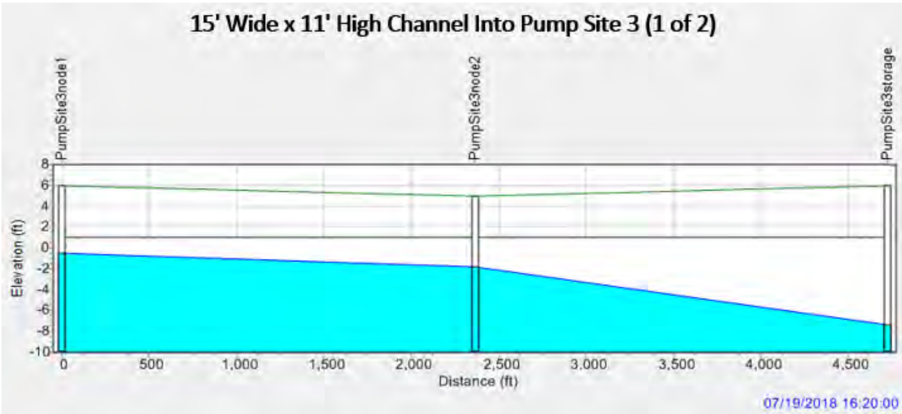
50-yr+0% rainfall – resulting water level timeseries in Offatts Bayou:

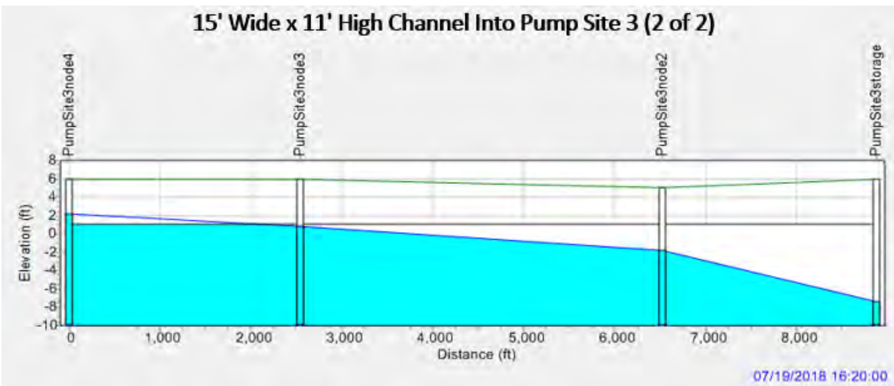


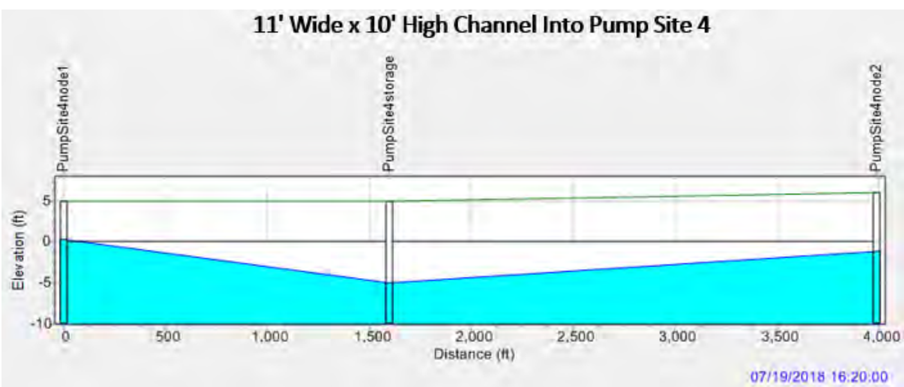
50-yr+0% rainfall – Profiles of peak water level timesteps within the channels:



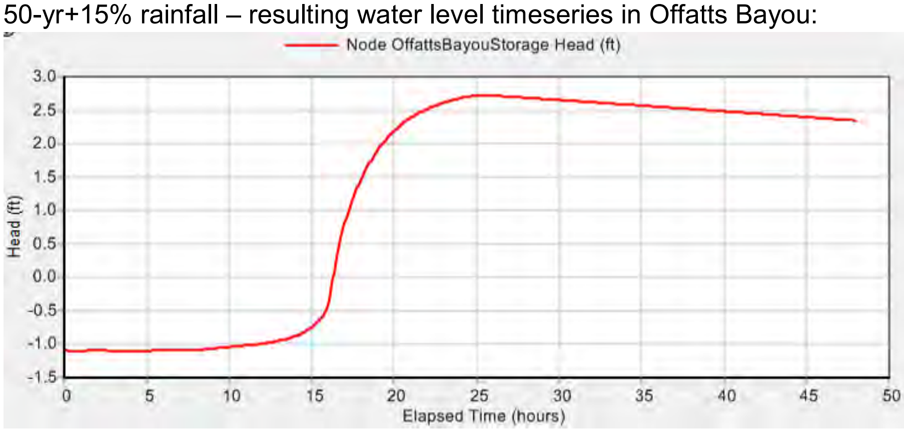




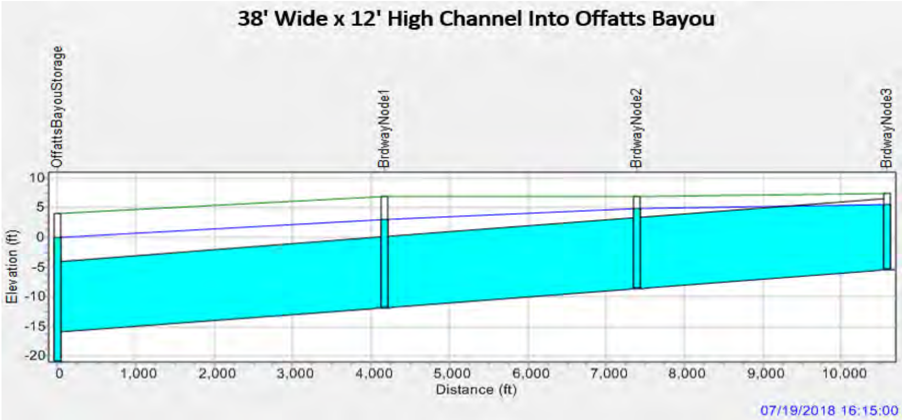


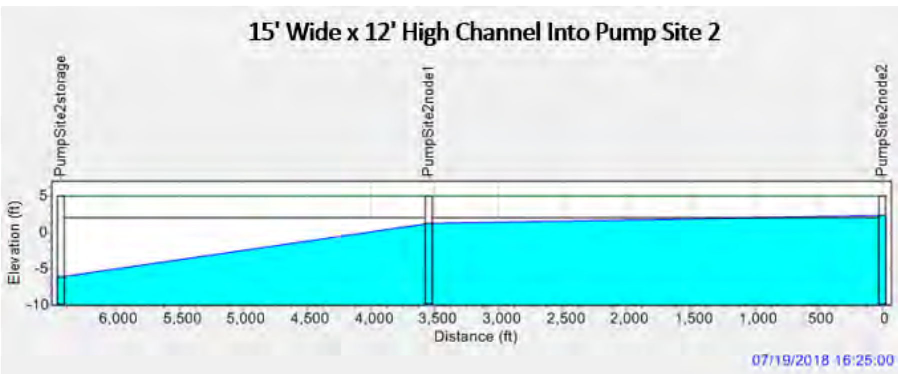


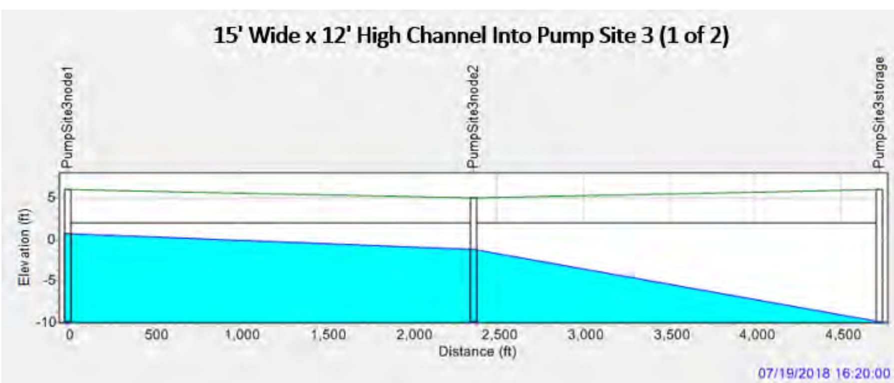
50-yr+15% rainfall – resulting water level timeseries in Offatts Bayou:

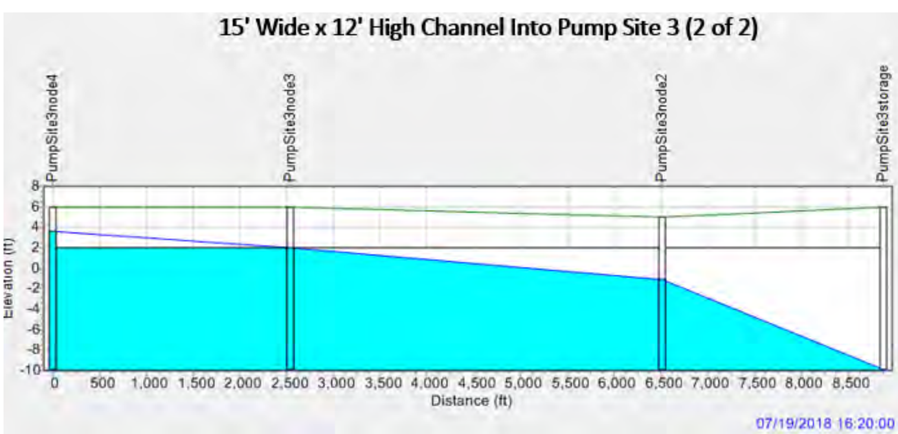


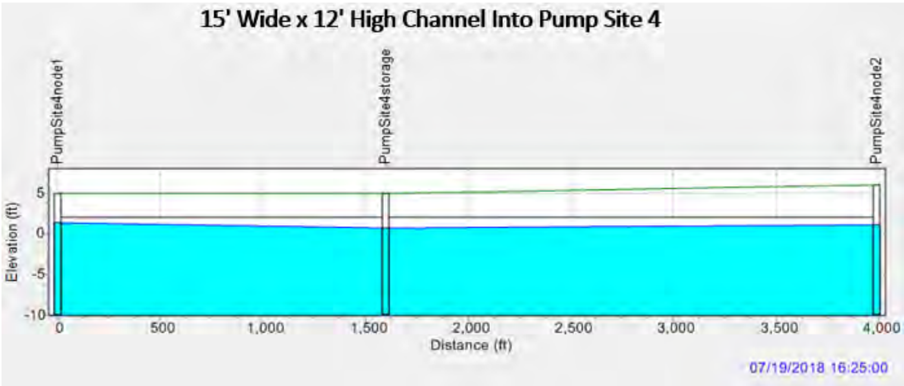
50-yr+15% rainfall – Profiles of peak water level timesteps within the channels:



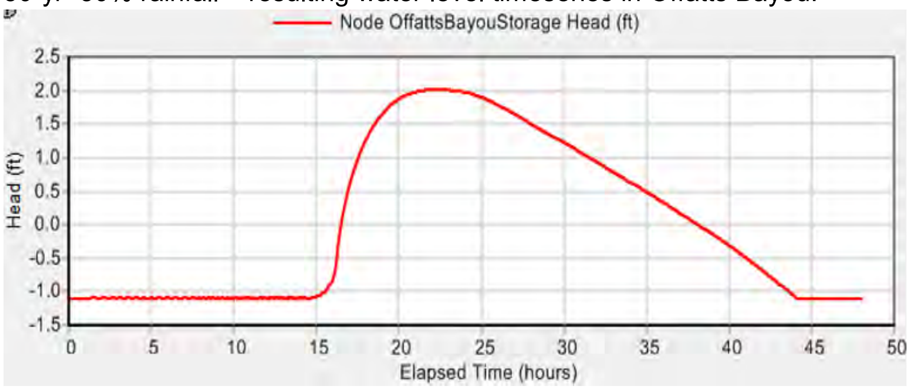




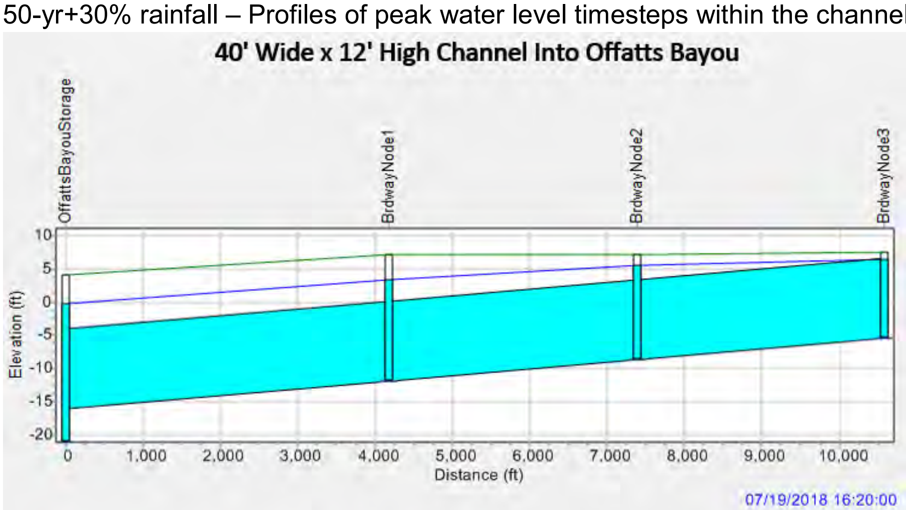


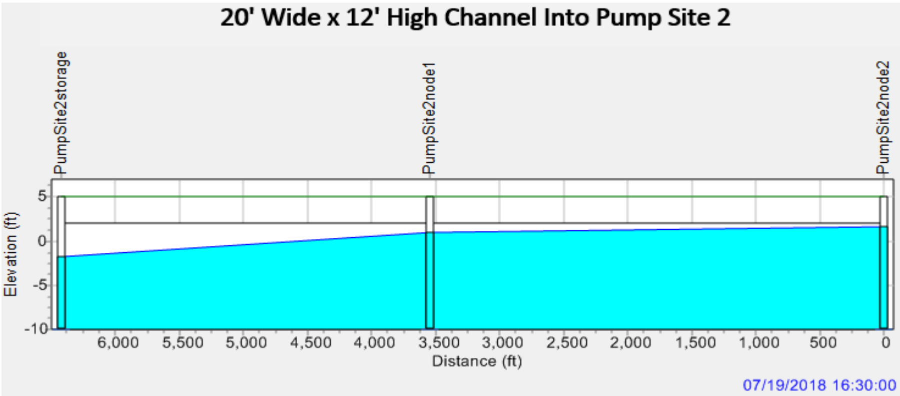


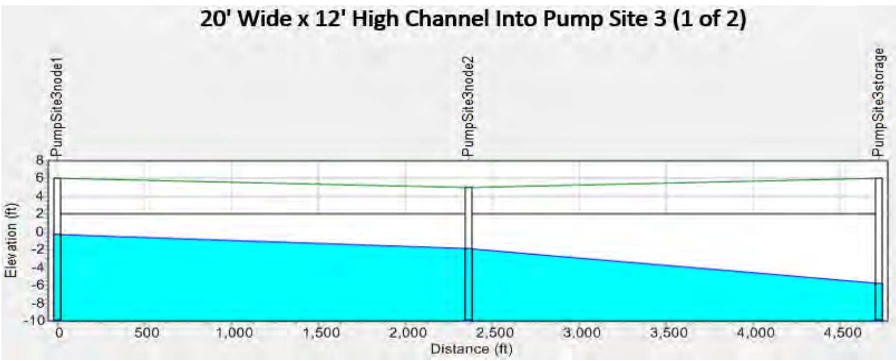
50-yr+30% rainfall – resulting water level timeseries in Offatts Bayou:

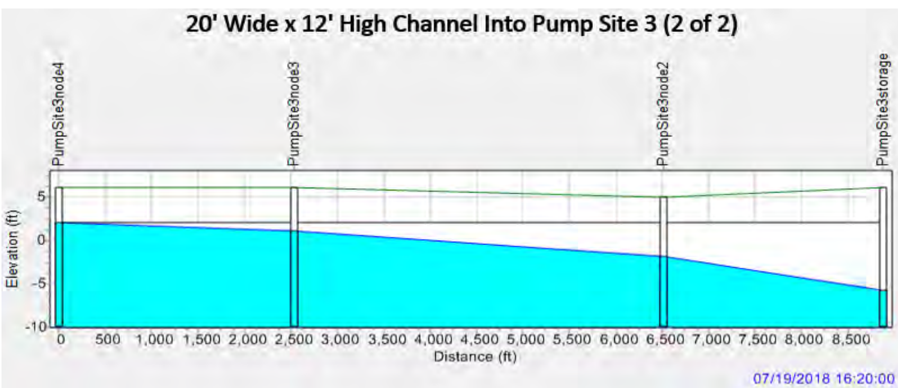


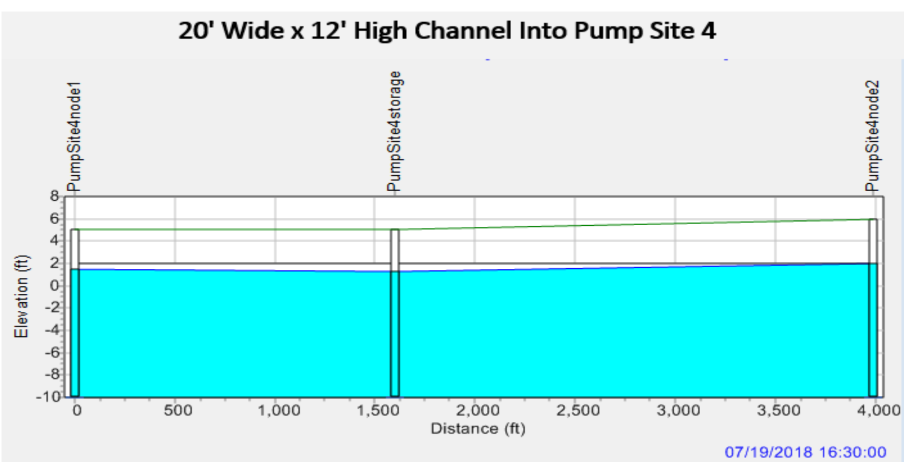
50-yr+30% rainfall – Profiles of peak water level timesteps within the channels:



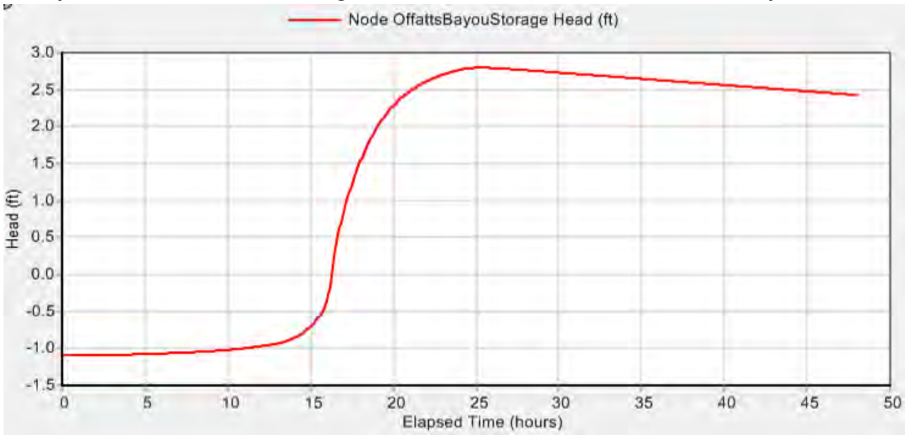

20' Wide x 12' High Channel Into Pump Site 2
PumpSite2storage
PumpSite2node1
PumpSite2node2
Elevation (ft)
Distance (ft)
07/19/2018 16:30:00


20' Wide x 12' High Channel Into Pump Site 3 (1 of 2)
PumpSite3node1
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)

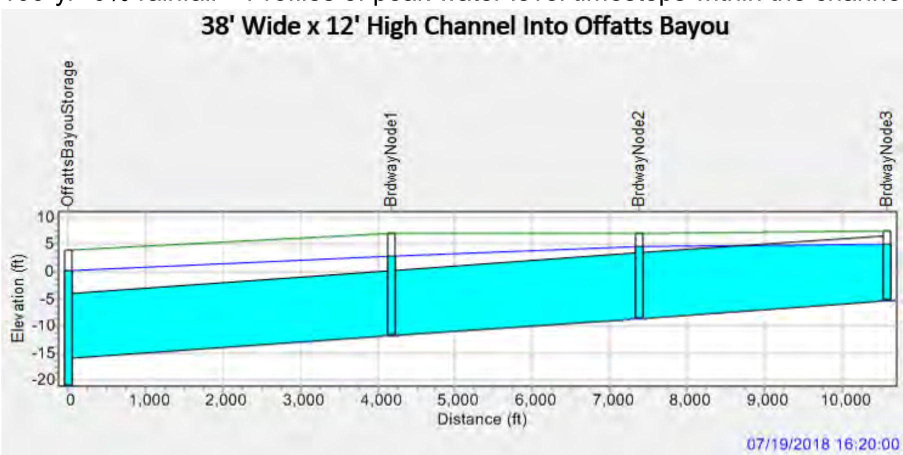

20' Wide x 12' High Channel Into Pump Site 3 (2 of 2)
PumpSite3node4
PumpSite3node3
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:20:00

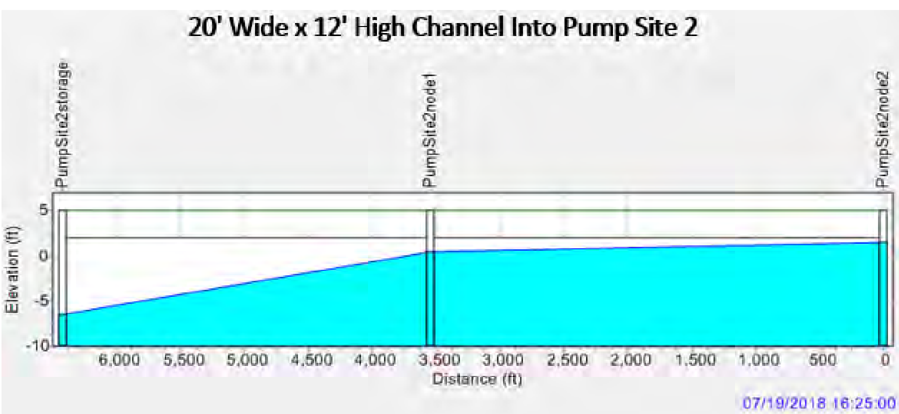

20' Wide x 12' High Channel Into Pump Site 4
PumpSite4node1
PumpSite4storage
PumpSite4node2
Elevation (ft)
Distance (ft)
07/19/2018 16:30:00

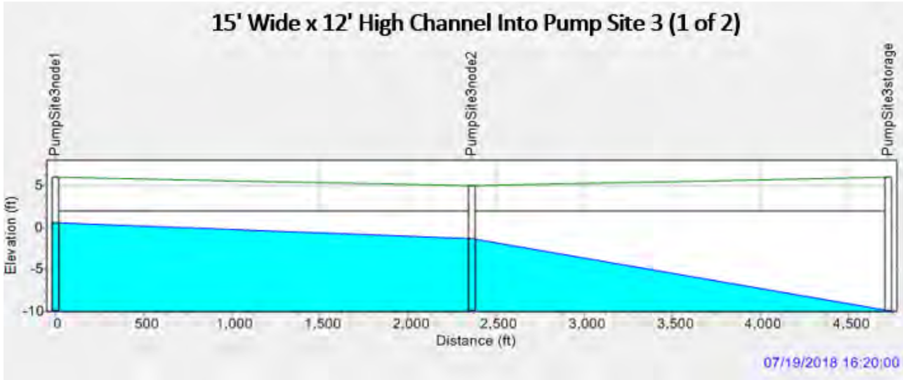
100-yr+0% rainfall – resulting water level timeseries in Offatts Bayou:

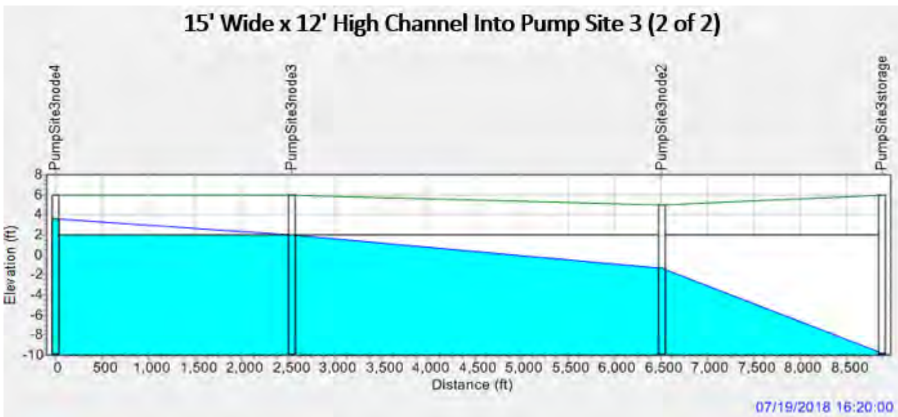


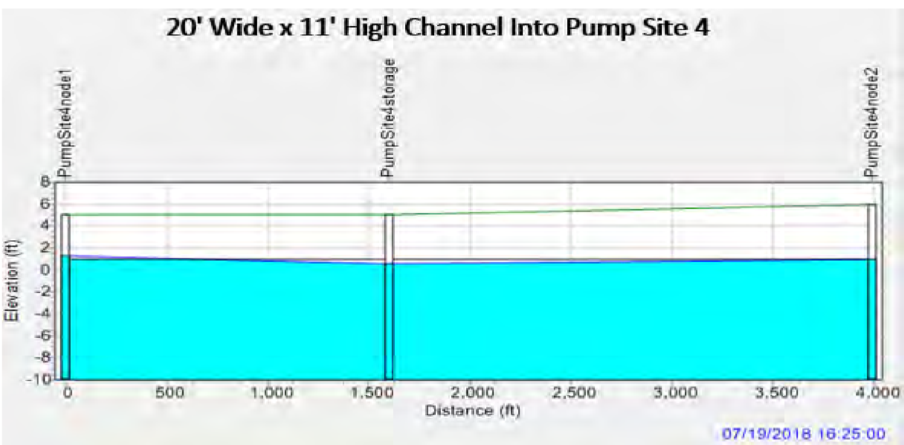
100-yr+0% rainfall – Profiles of peak water level timesteps within the channels:



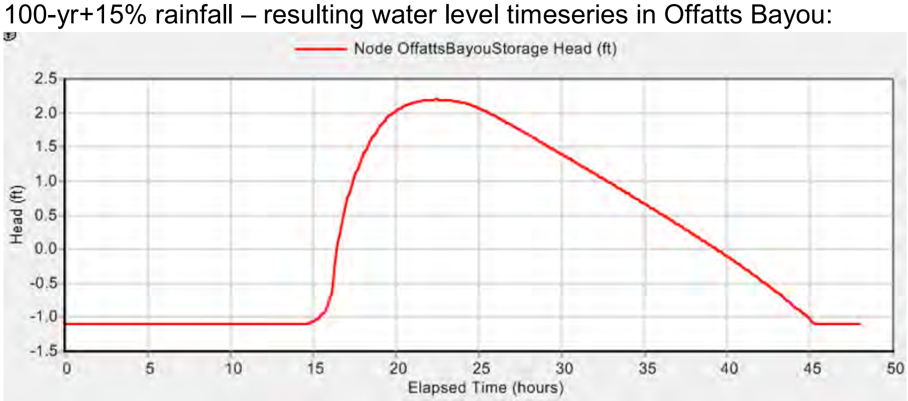




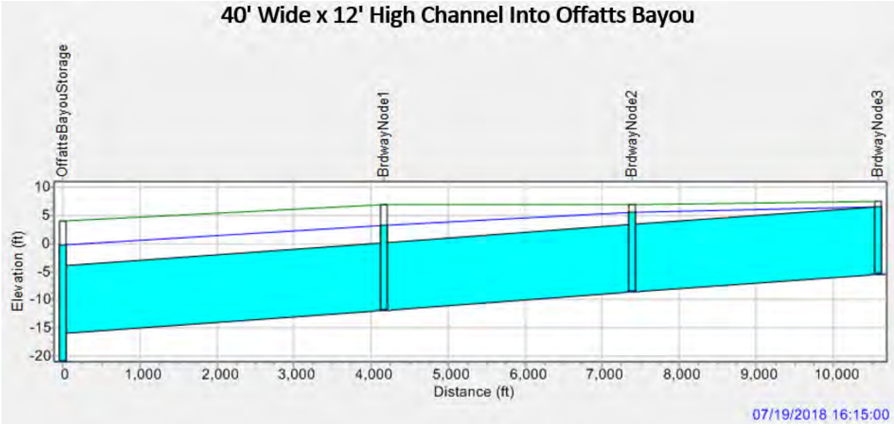


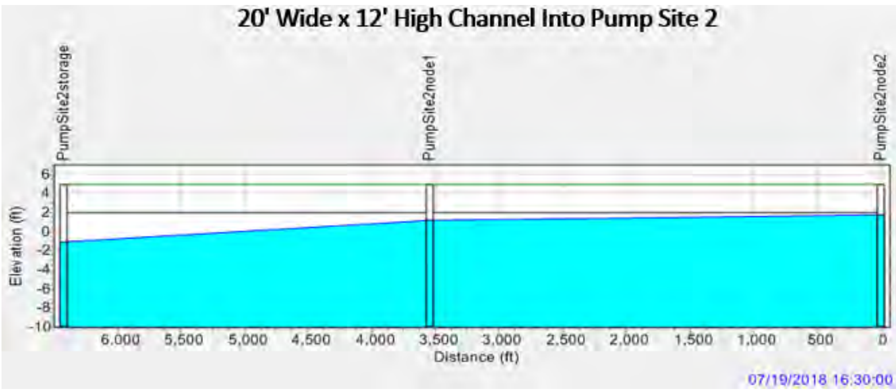


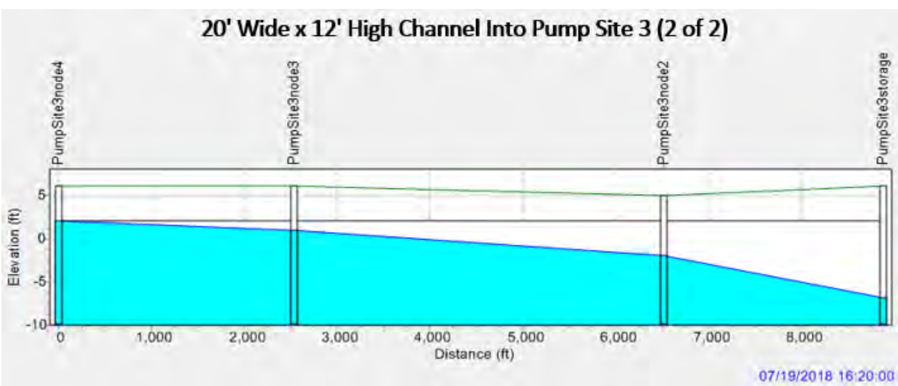
100-yr+15% rainfall – resulting water level timeseries in Offatts Bayou:

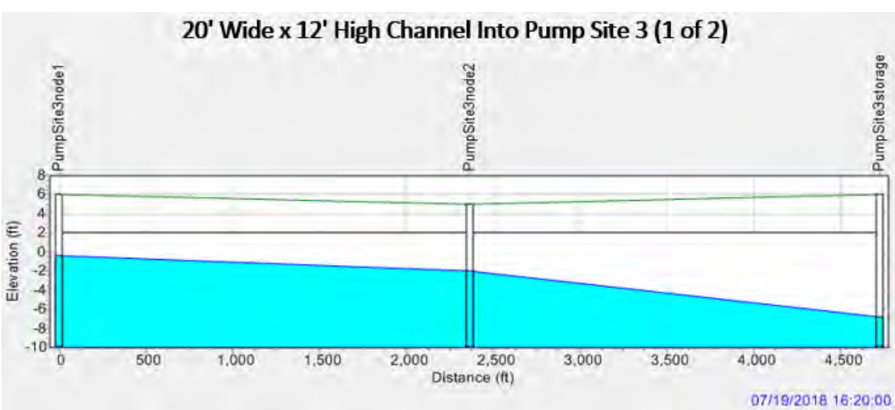


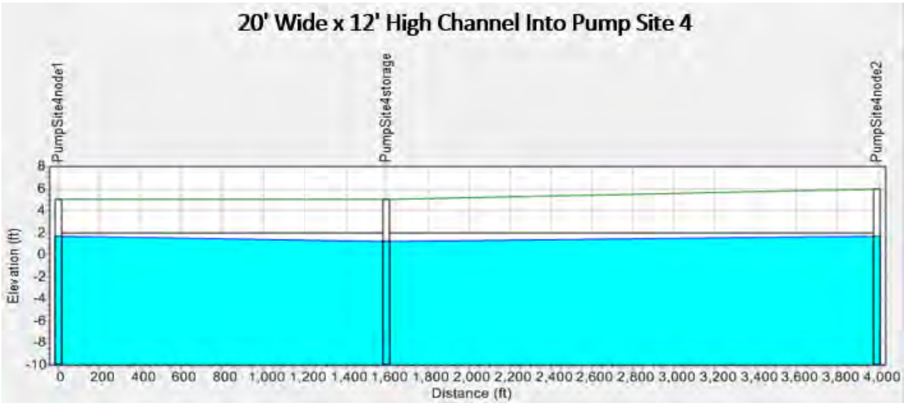
100-yr+15% rainfall – Profiles of peak water level timesteps within the channels:



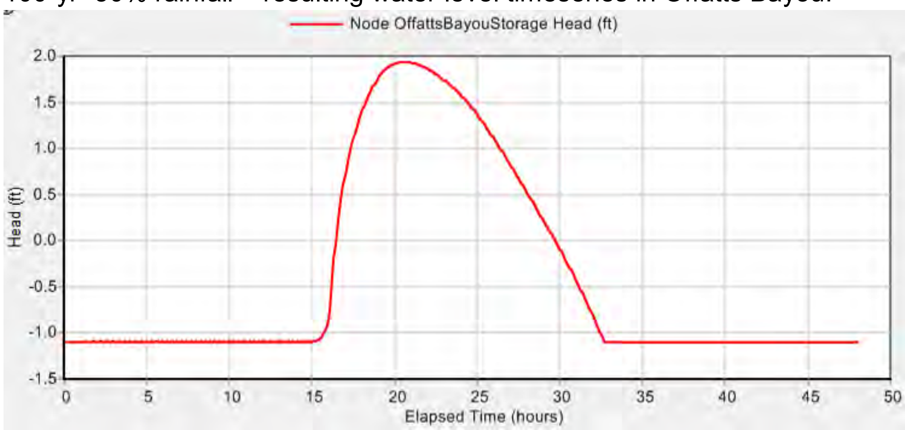




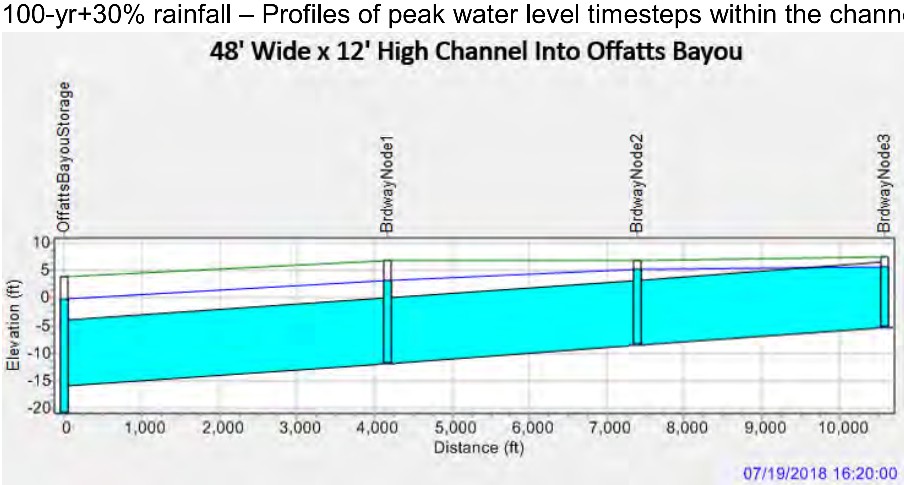


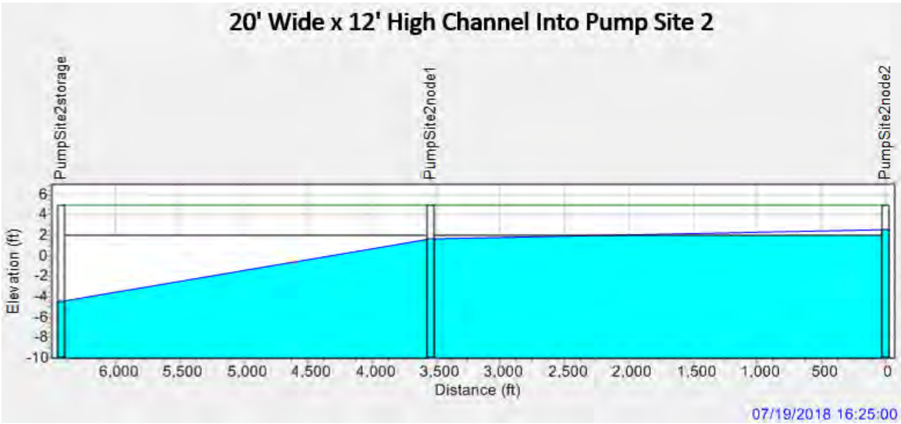


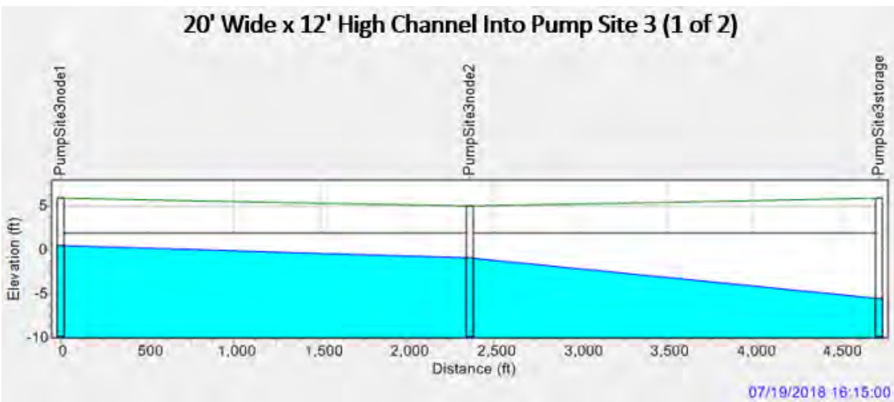
100-yr+30% rainfall – resulting water level timeseries in Offatts Bayou:



100-yr+30% rainfall – Profiles of peak water level timesteps within the channels:




20' Wide x 12' High Channel Into Pump Site 2
PumpSite2storage
PumpSite2node1
PumpSite2node2
Elevation (ft)
Distance (ft)
07/19/2018 16:25:00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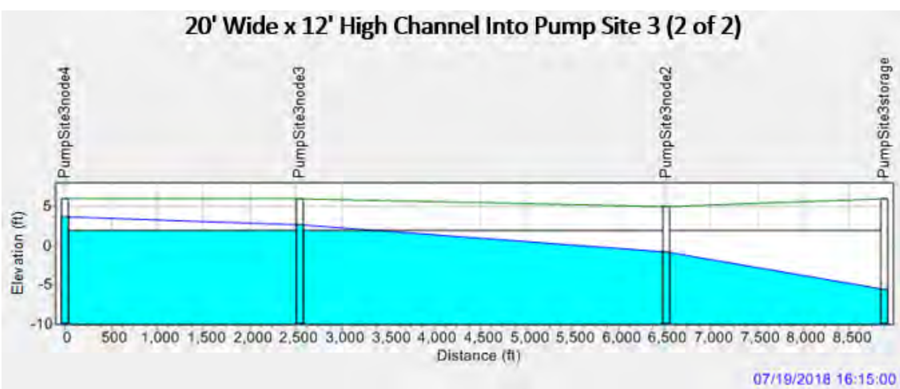
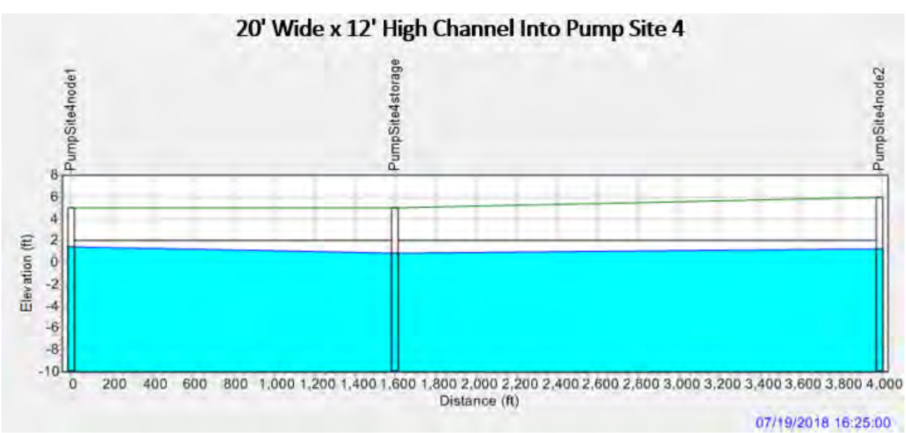
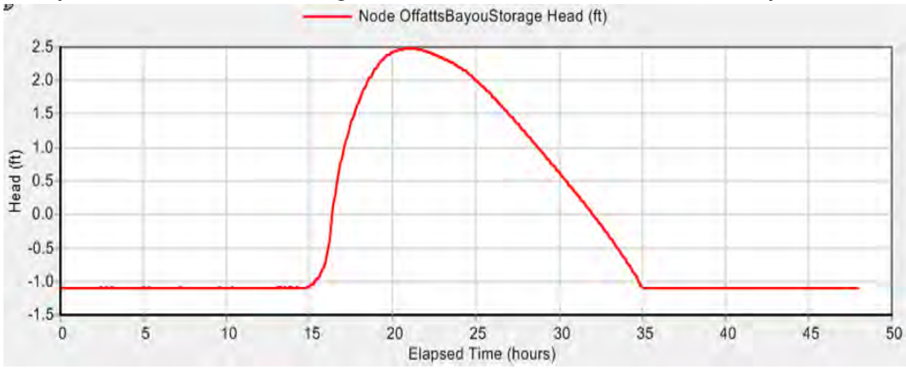
PumpSite3node1
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:15:00

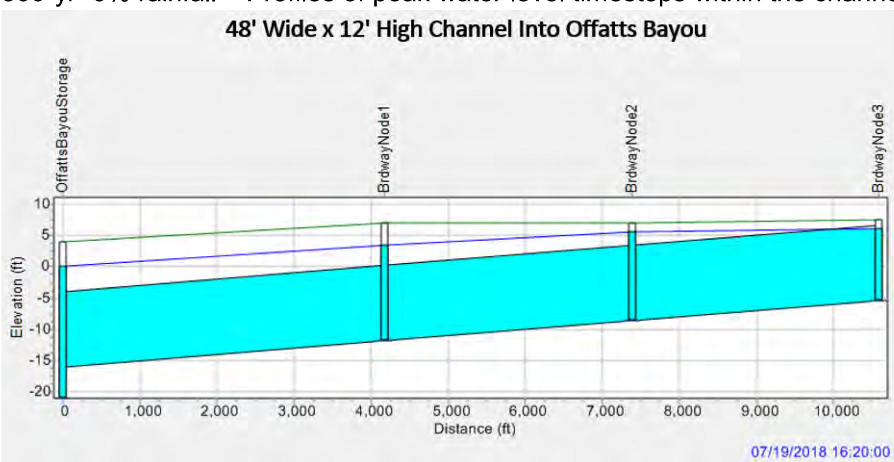

20' Wide x 12' High Channel Into Pump Site 3 (2 of 2)
PumpSite3node4
PumpSite3node3
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:15:00

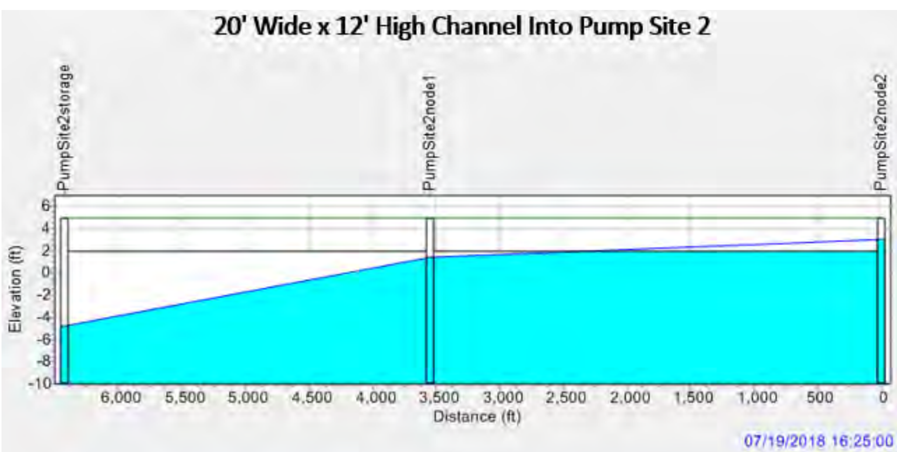

20' Wide x 12' High Channel Into Pump Site 4
PumpSite4node1
PumpSite4storage
PumpSite4node2
Elevation (ft)
Distance (ft)
07/19/2018 16:25:00

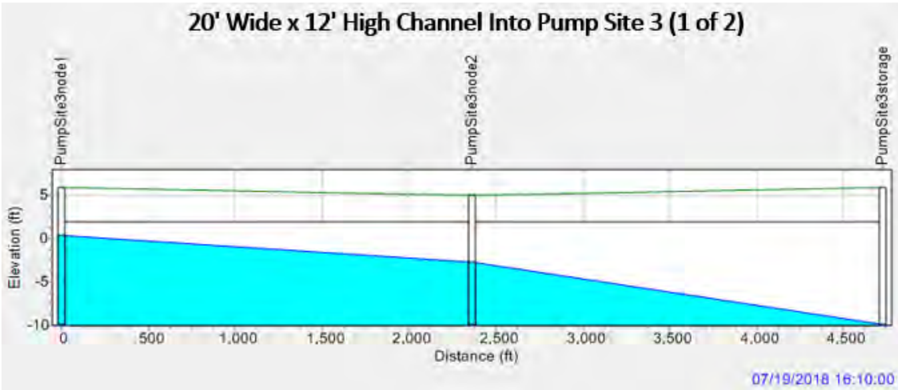
500-yr+0% rainfall – resulting water level timeseries in Offatts Bayou:

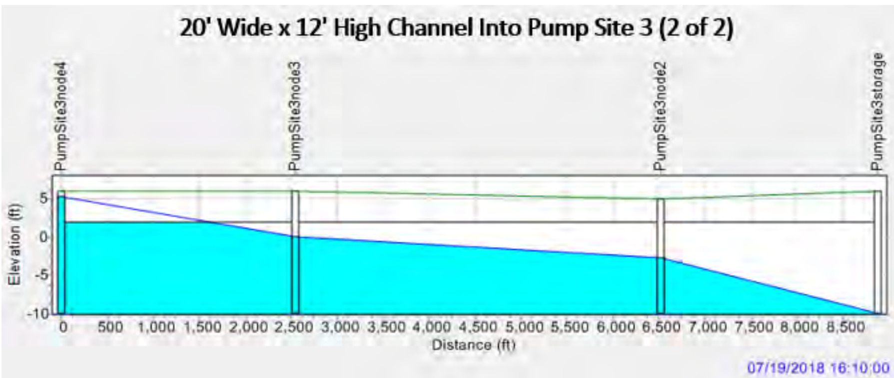


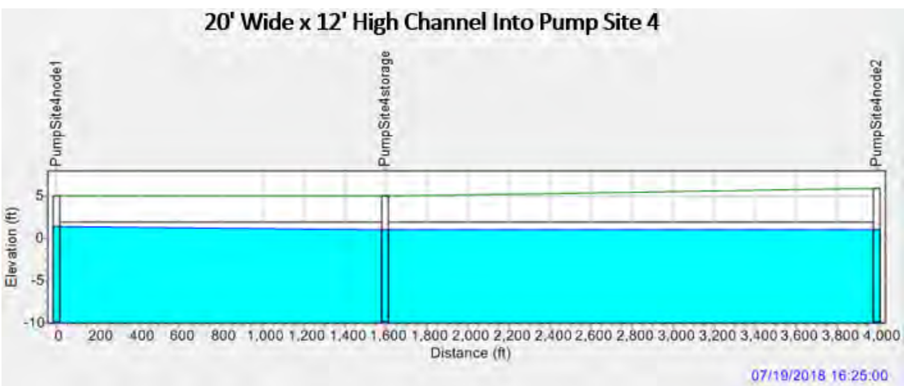
500-yr+0% rainfall – Profiles of peak water level timesteps within the channels:



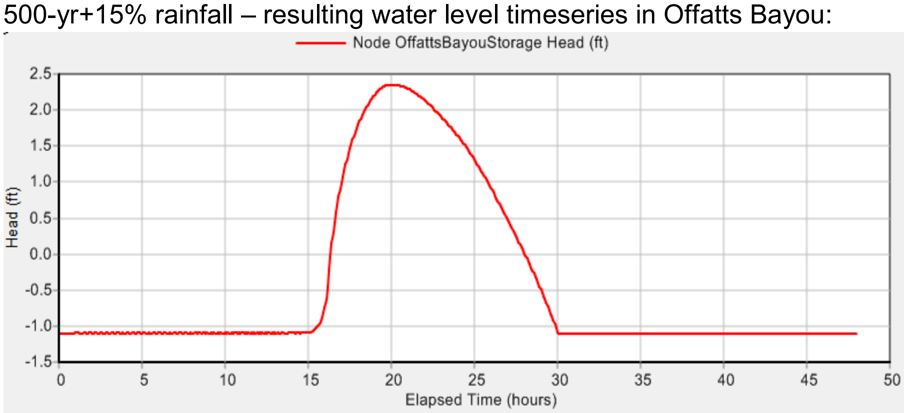




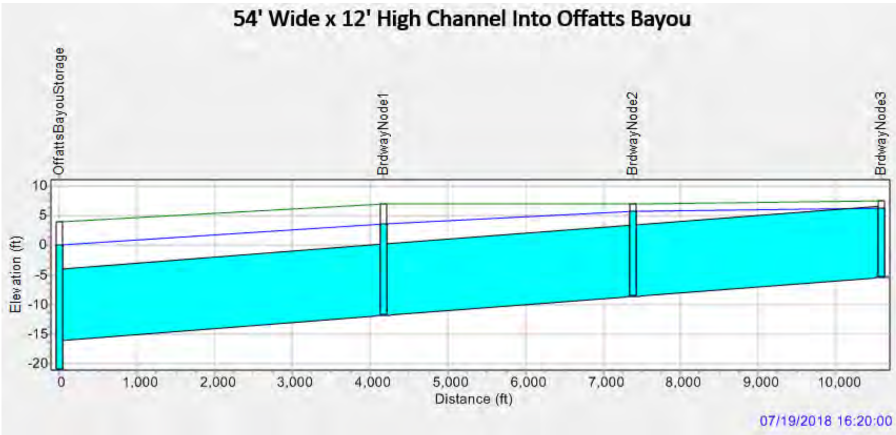


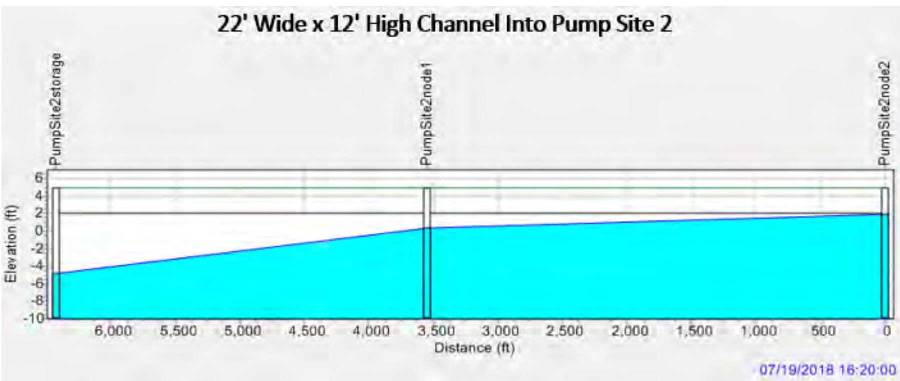


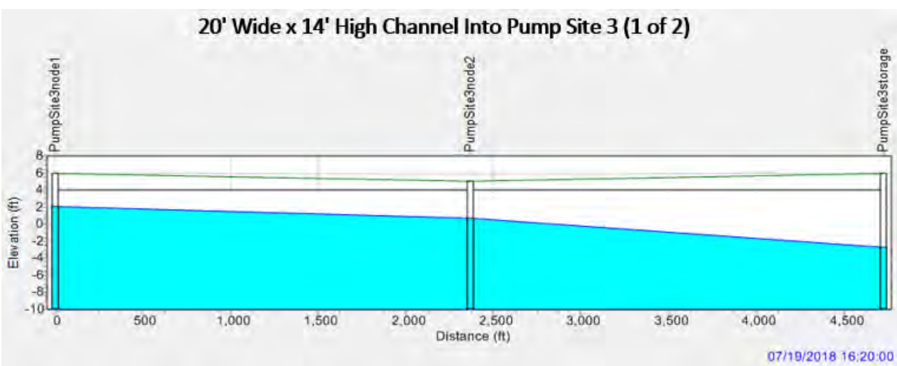
500-yr+15% rainfall – resulting water level timeseries in Offatts Bayou:

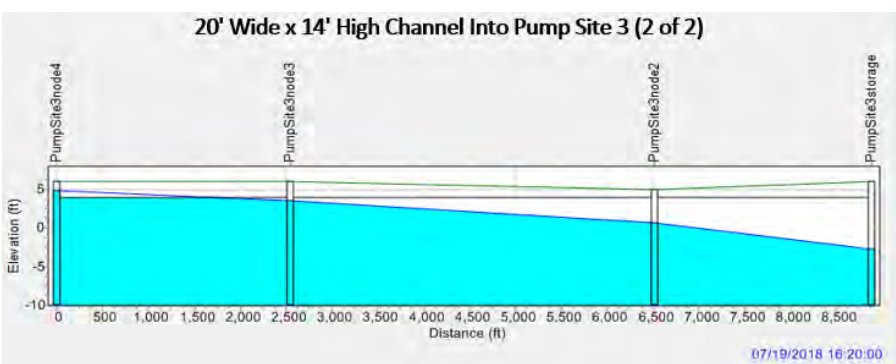


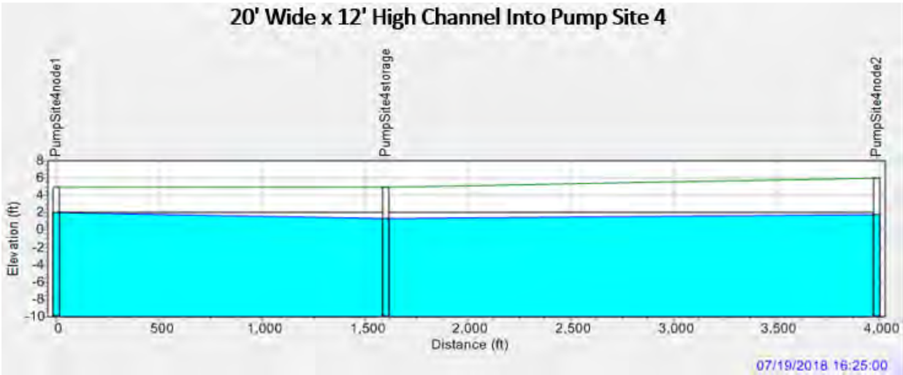
500-yr+15% rainfall – Profiles of peak water level timesteps within the channels:



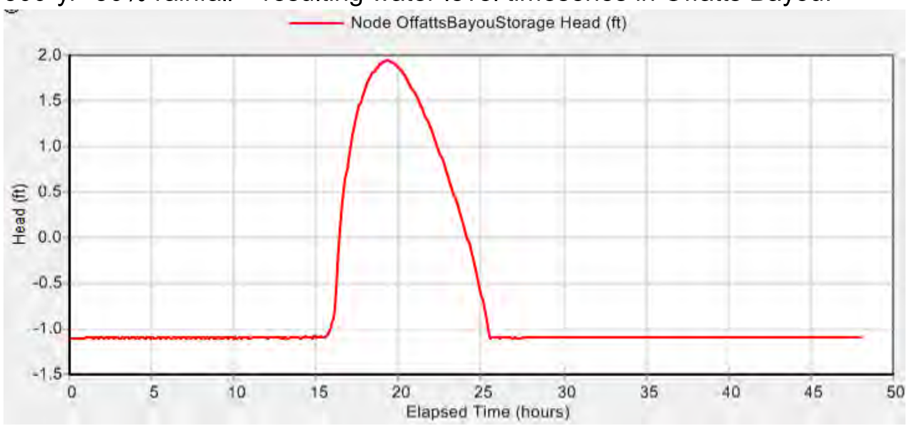




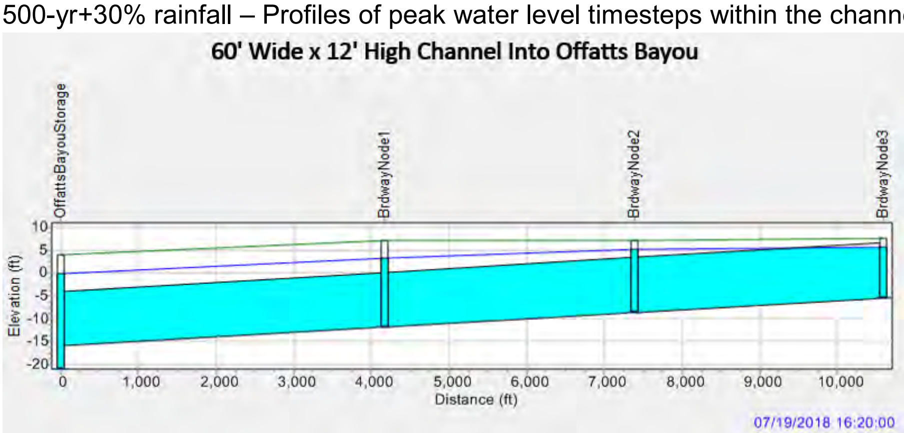


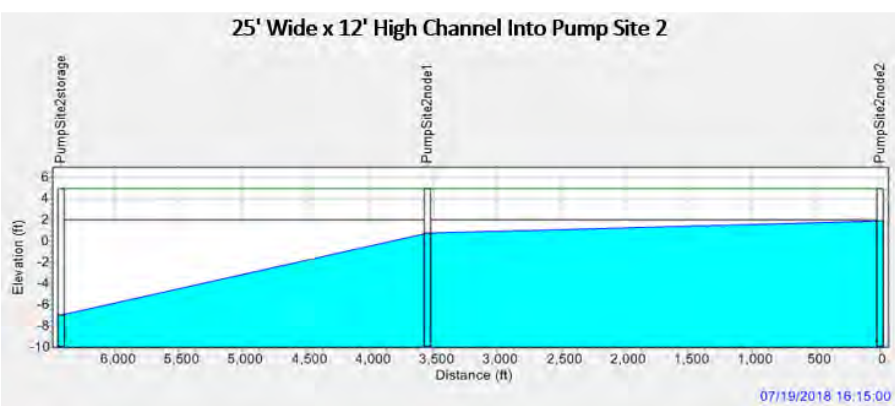


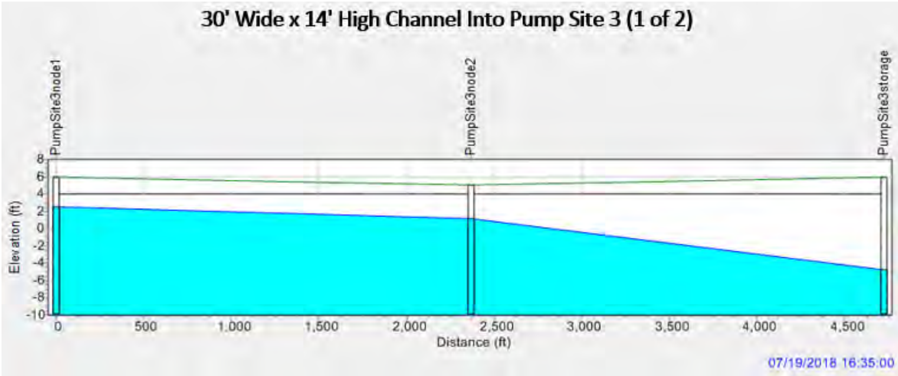
500-yr+30% rainfall – resulting water level timeseries in Offatts Bayou:

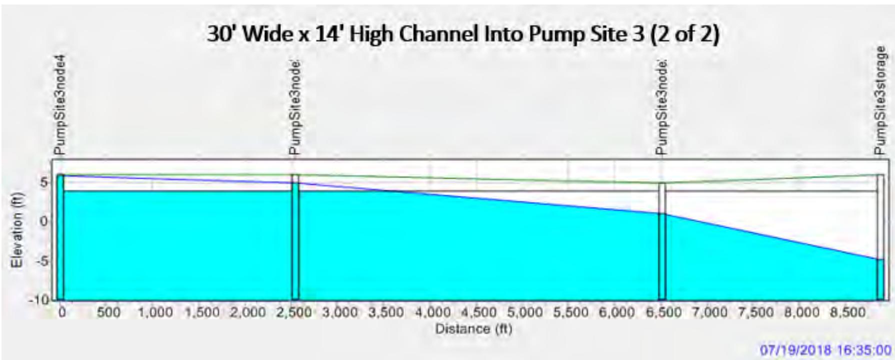


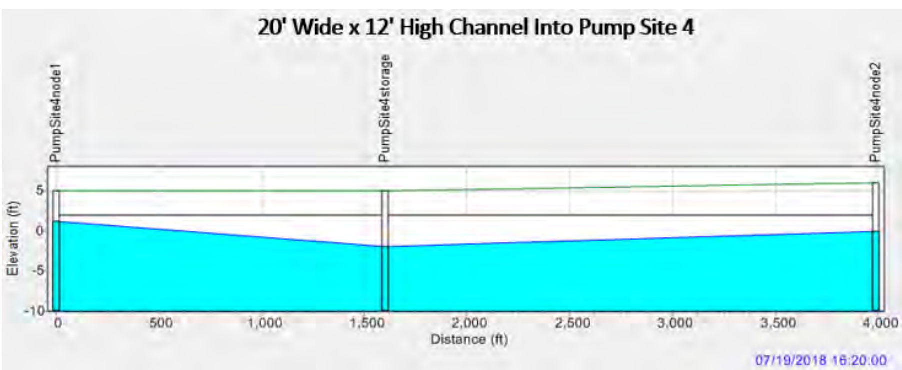
500-yr+30% rainfall – Profiles of peak water level timesteps within the channels:






30' Wide x 14' High Channel Into Pump Site 3 (1 of 2)
PumpSite3node1
PumpSite3node2
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:35:00


30' Wide x 14' High Channel Into Pump Site 3 (2 of 2)
PumpSite3node4
PumpSite3storage
Elevation (ft)
Distance (ft)
07/19/2018 16:35:00


20' Wide x 12' High Channel Into Pump Site 4
PumpSite4node1
PumpSite4storage
PumpSite4node2
Elevation (ft)
Distance (ft)
07/19/2018 16:20:00

Appendix C. Documented Changes to Hydrologic and Hydraulic Models for Clear Creek Watershed

C.1 Extrapolation of Inflow-Diversion and Storage-Discharge Tables

As discussed in Section 5.2, 7 inflow-diversion tables and 22 storage-discharge tables were extrapolated to accommodate the heavy rainfall intensity of a 500-yr (+30%) event. These changes are documented below. The extrapolated value is indicated in red for the extrapolation tables and figures.

Table C 1. Summary of Storage-Discharge extrapolation.

Reach	Table No.	Extrapolated Storage [acre-feet]	Extrapolated Discharge [cfs]
A1000000_2385R	353	1,500.0	3,314.3
A1000000_2199R	356	3,500.0	7,263.0
A1000000_2147R	357	4,500.0	6,165.1
A1000000_1793R	359	15,000.0	10,168.9
A1000000_1296R	362	20,000.0	11,412.3
MARY0100_9902R	368	5,000.0	7,085.9
MARY0100_9901R	369	2,000.0	8,388.1
COWA0103_9904R	371	1,500.0	6,720.7
COWA0100_9902R	373	1,000.0	8,327.9
COWA0100_9901R	374	1,800.0	11,503.3
CHIG0100_9901R	380	1,000.0	12,408.8
CG000100_9901R	386	40.0	2,216.2
MAGN0100_9901R	387	200.0	10,580.1
B1000000-0628R	395	400.0	5,673.2
B1000000-0483R	399	2,000.0	22,440.4
B1000000-0364R	400	3,000.0	22,245.6
B1000000-0265R	401	4,000.0	26,562.4
B1000000-0222R	402	3,000.0	36,032.2
B1000000-0149R	403	3,500.0	40,556.3
B1000000-0021R	421	6,500.0	57,344.1
A1000000_0161R	425	12,000.0	251,037.6
A1000000_0000R	426	20,000.0	129,450.8

Table C 2. Summary of Inflow-Diversion extrapolation.

Diversion	Table No.	Extrapolated Inflow [cfs]	Extrapolated Diversion [cfs]
B1040400-0040D	53	2,500	1,288.6
COWA0100_9902D	54	3,000	316.0
COWA0102_9901D	57	4,000	474.0
A1040000-0296D	60	5,000	4,247.5
B1090000-0016D	61	3,000	1,895.6
HICK0100_9901D	64	15,000	239.0
COWA0100_9901D	65	4,000	963.0
HICK0100_9901_D	129	15,000	239.0

Most of the Inflow-Diversion tables were extrapolated using either a simple linear or polynomial regression on all available data. Figure C 1 shows an example extrapolation of the Inflow-Diversion Table for Diversion A1040000-0296D, the data for which is presented in Table C 3.

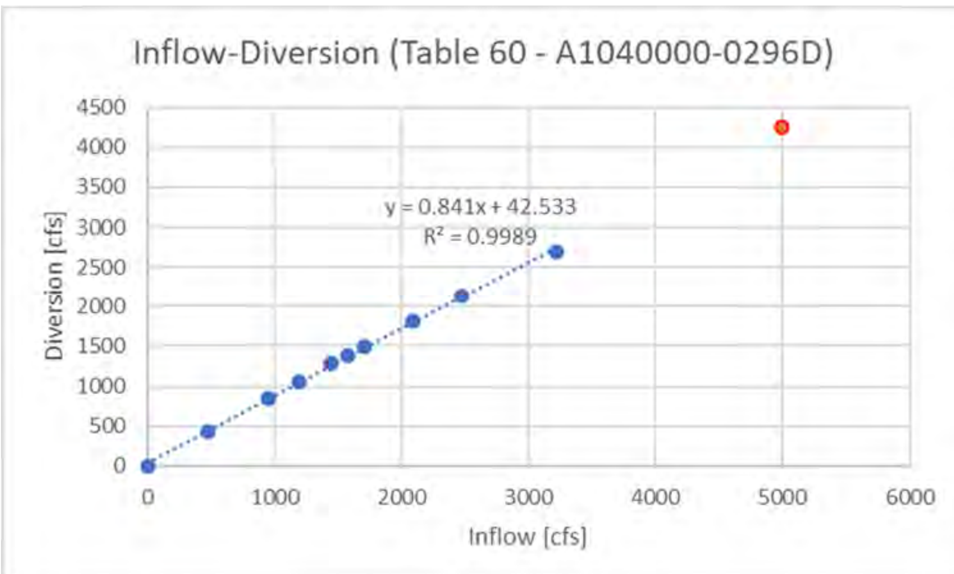


Figure C 1. Inflow-Diversion extrapolation plot for Table-60 at diversion A1040000-0296D

Table C 3. Inflow-Diversion extrapolation table for Table-60 at diversion A1040000-0296D.

Inflow [cfs]	Diversion [cfs]
0	0
473.5	426
947	852.9
1196	1063.9
1445	1276
1575.5	1386
1706	1496
2092	1815.9

Inflow [cfs]	Diversion [cfs]
2478	2136.9
3221.4	2700
5000	4247.5

For some Inflow-Diversion tables, the diversion reaches a maximum capacity at which it becomes constant. For these scenarios, the constant value was maintained for the extrapolation as shown in Figure C 2 and presented in Table C 4 for diversion COWA0102_9901D.

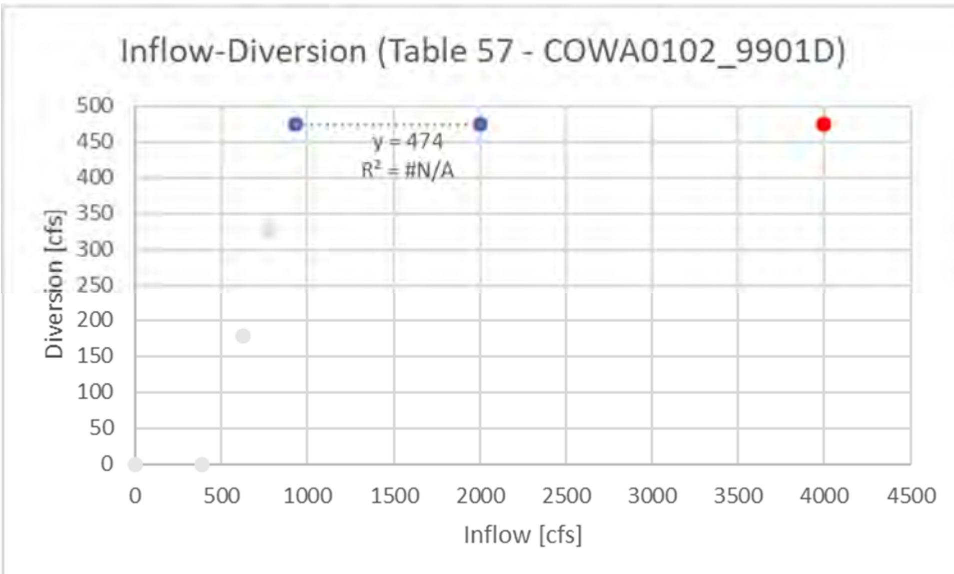


Figure C 2. Inflow-Diversion extrapolation plot for Table-57 at diversion COWA0102_9901D.

Table C 4. Inflow-Diversion extrapolation table for Table-57 at diversion COWA0102_9901D.

Inflow [cfs]	Diversion [cfs]
0	0
388	0
621	179
776	326
931	474
2000	474
4000	474

Many of the Storage-Discharge tables see a logarithmic trend between storage and discharge for low storage values and a linear trend for larger values. As shown in Figure C 3 and presented in Table C 5 for reach B1000000-0628R, these tables were only extrapolated using the linear data.

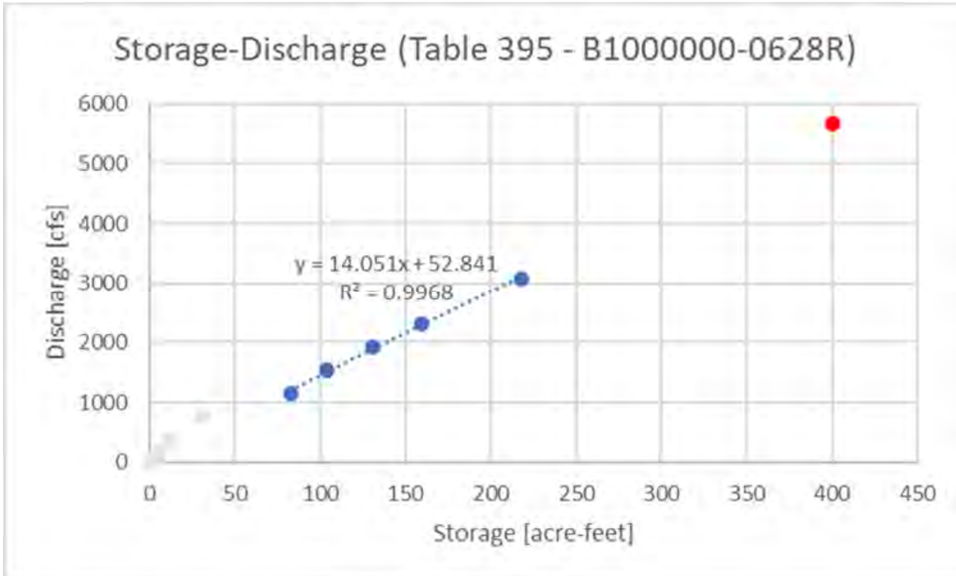
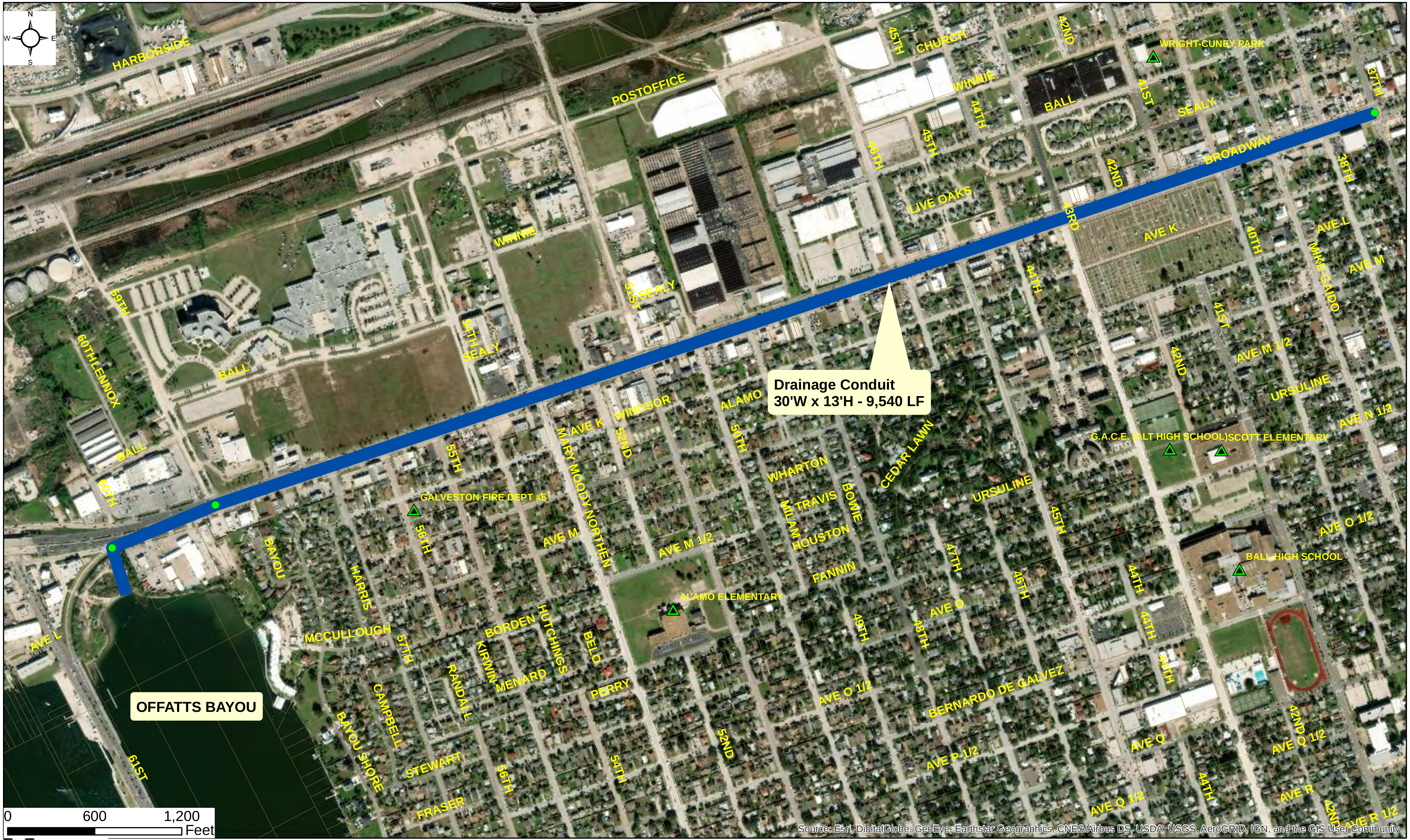


Figure C 3. Storage-Discharge extrapolation plot for Table-395 at reach B1000000-0628R.

Table C 5. Storage-Discharge extrapolation table for Table-395 at reach B1000000-0628R.

Storage [acre-feet]	Discharge [cfs]
0	0
3.7	96
6.3	193
11.7	385
30.9	771
82.7	1156
103.5	1541
131.1	1926
159.1	2312
217.7	3082
217.7	3082
400	5673.2

Appendix D. Galveston Concept Plans



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Proposed Stormwater Conduit
For Offats Bayou Pump Station #1
City of Galveston, Texas.

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Proposed Stormwater Conduit
and Potential Pump Station Site #2
City of Galveston, Texas.

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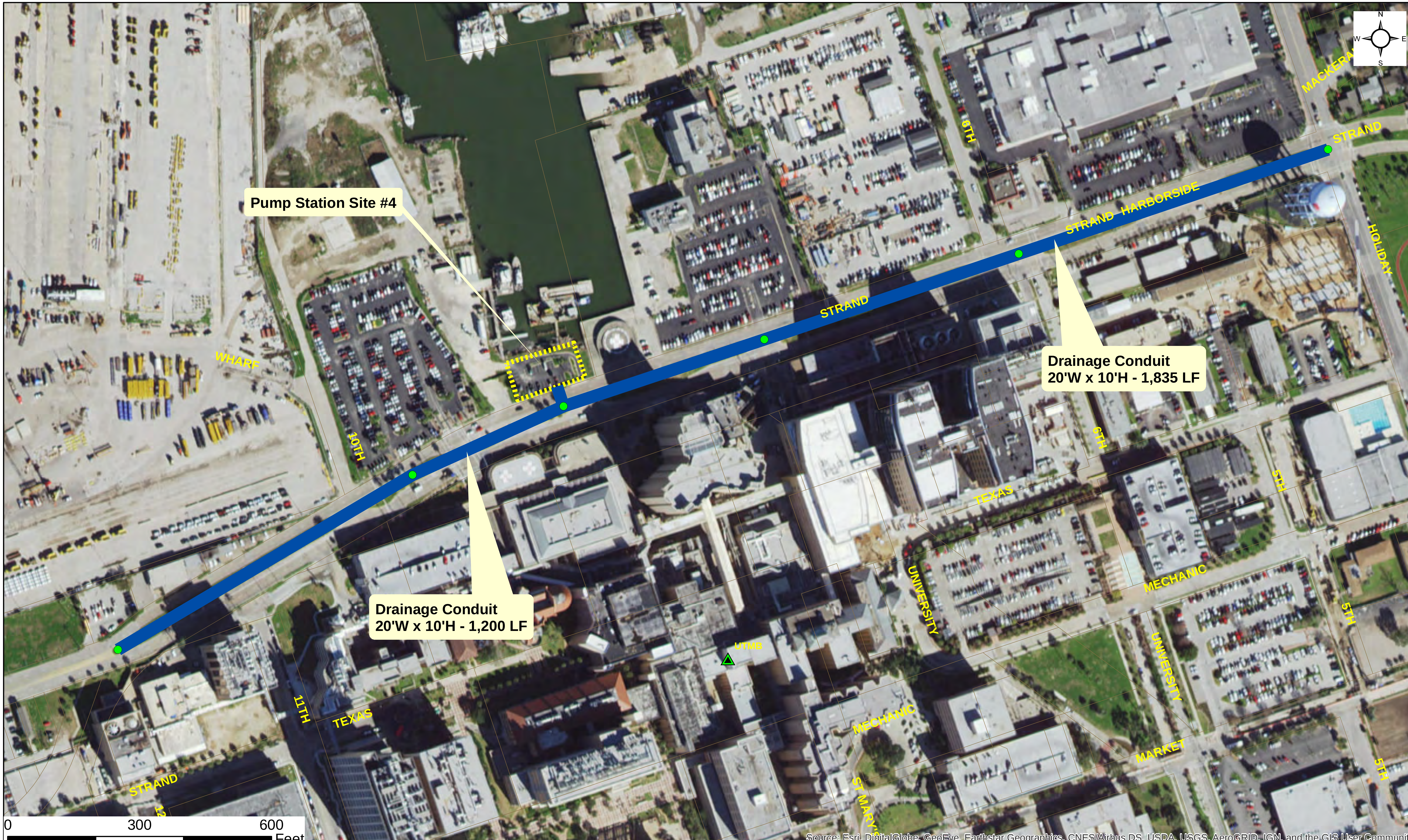
Proposed Stormwater Conduit
and Potential Pump Station Site #3
City of Galveston, Texas.



MOTT MACDONALD

Proposed Stormwater Conduit
and Potential Pump Station Site #4
City of Galveston, Texas.

Appendix E. Galveston Conduit Plan and Profiles

PLAN VIEW

SCALE 1"=100'

Pump Station #1 Collection Conduits
STA 0+00 TO 32+33

PROFILE VIEW

H. SCALE 1"=100'
V. SCALE 1"=10'

MOTT MACDONALD

111 Wood Avenue South
Iselin NJ 08830-4112
United States of America
Certificate No. 24GA28016600

T +1 (800) 832 3272
F +1 (973) 376 1072
www.mottmacamericas.com

Client: USACE

Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security STD

Project Number 393582 | B/O | Total | Drawing Number

Title
TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#1
SHEET 1 OF 3

PLAN VIEW

SCALE 1"=100'

Pump Station #1 Collection Conduits
STA 32+33 TO 64+66

PROFILE VIEW

H. SCALE 1"=100'
V. SCALE 1"=10'

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F +1 (973) 376 1072
www.mottmacamericas.com

Client: USACE

Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

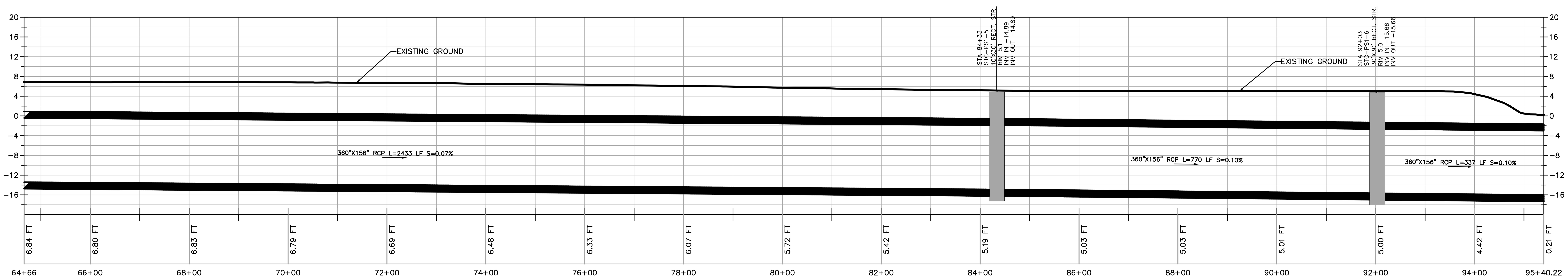
Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security: STD

Project Number: 393582 | B/O | Total | Drawing Number

Title

TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#1
SHEET 2 OF 3

PLAN VIEW
SCALE 1"=100'



Pump Station #1 Collection Conduits
STA 64+66 TO 95+40

PROFILE VIEW
H. SCALE 1"=100'
V. SCALE 1"=10'

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111 Wood Avenue South
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United States of America
Certificate No. 24GA28016600

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Client: USACE

Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security STD

Project Number: 393582 | B/O | Total

Drawing Number

Title
TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#1
SHEET 3 OF 3

PLAN VIEW

SCALE 1"=100'

Pump Station #3 Collection Conduits East Side
STA 0+00 TO 32+33

PROFILE VIEW

H. SCALE 1"=100'
V. SCALE 1"=10'

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Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

Designed	ZMS		Eng check		
Drawn	ZMS		Coordination		
Dwg check			Approved		
Scale at ANSI D	Status		Rev	Security: STD	
Drawing Number					

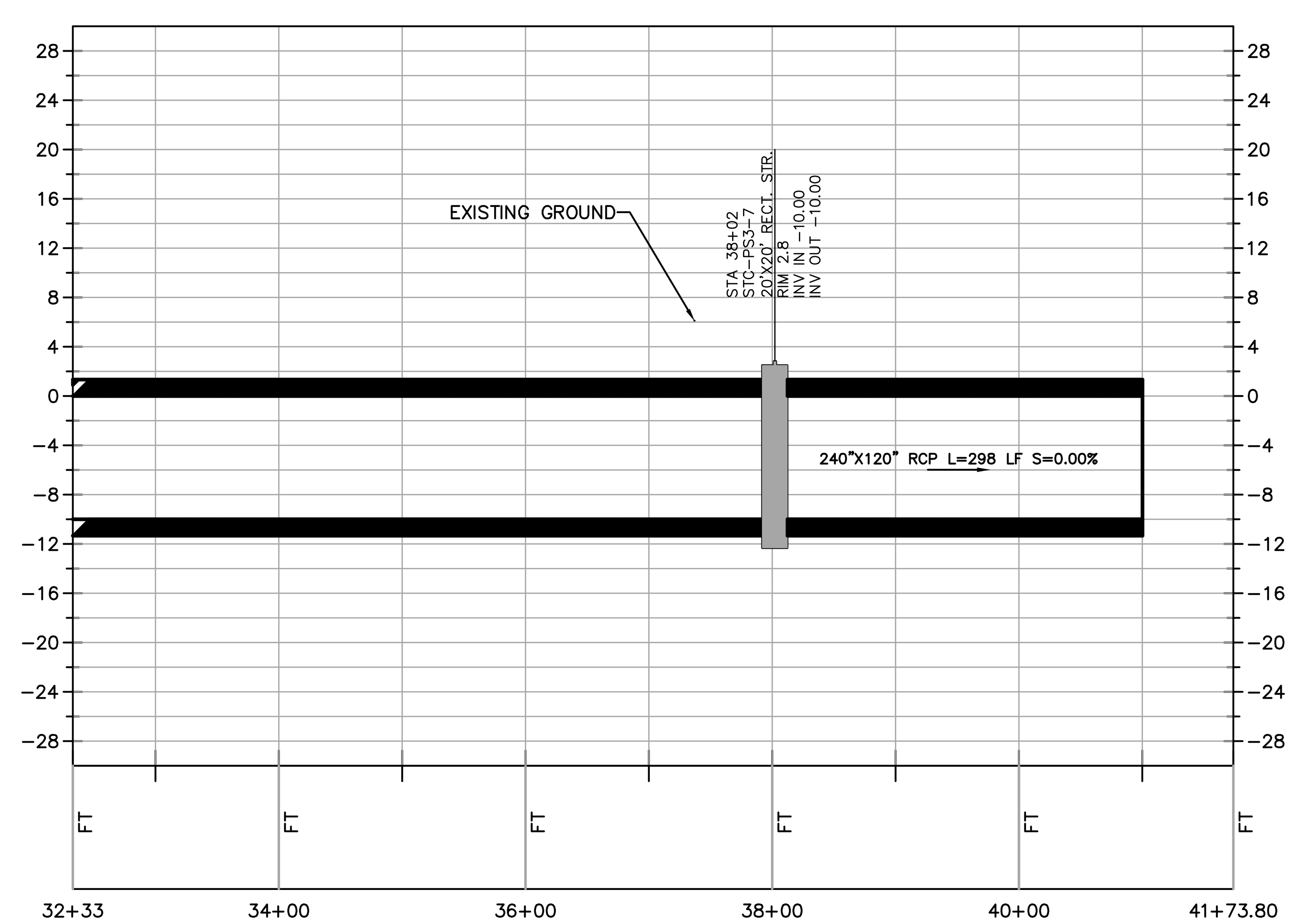
Project Number	B/O	Total
393582		

Title

TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#3
SHEET 1 OF 4

PLAN VIEW

SCALE 1"=100'



Pump Station #3 Collection Conduits East Side

STA 32+33 TO 41+74

PROFILE VIEW

H. SCALE 1"=100'

V. SCALE 1"=10'

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Client: USACE

Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	

Scale at ANSI D	Status	Rev	Security
			STD

Project Number	B/O	Total	Drawing Number
393582			

Title

TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#3
SHEET 2 OF 4

PROFILE VIEW
H. SCALE 1"=100'
V. SCALE 1"=10'

Pump Station #3 Collection Conduits South Side
STA 0+00 TO 32+33

M M
MOTT MACDONALD

111 Wood Avenue South
Iselin NJ 08830-4112
United States of America
Certificate No. 24GA28016600

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F +1 (973) 376 1072
www.mottmacamericas.com

Client: USACE

Rev	Date	Drawn	Description	Ch'k'd	App'd

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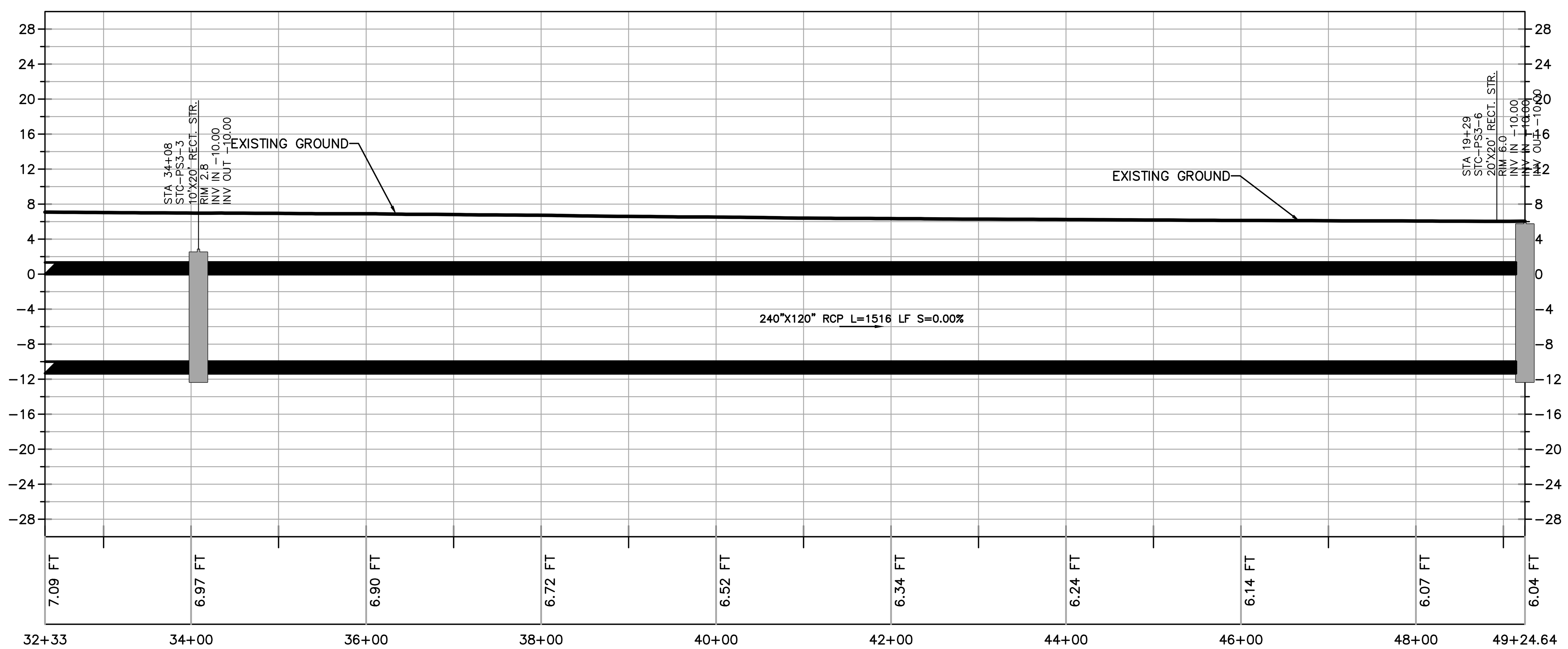
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security STD

Project Number	B/O	Total	Drawing Number
393582			

Title
TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#3
SHEET 3 OF 4

PLAN VIEW

SCALE 1"=100'



Pump Station #3 Collection Conduits South Side
STA 32+33 TO 49+25

PROFILE VIEW

H. SCALE 1"=100'
V. SCALE 1"=10'

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Date

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security STD

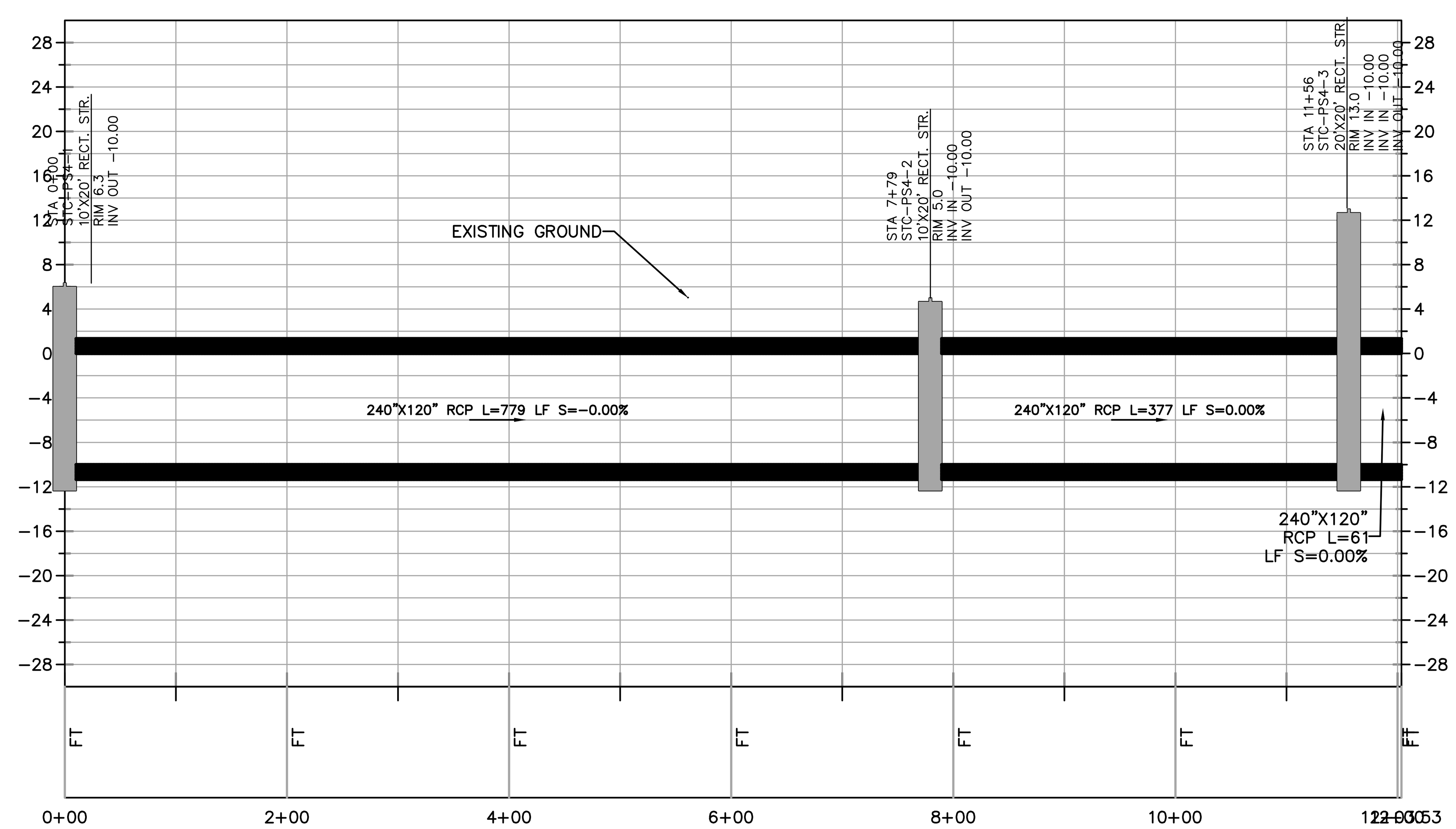
Project Number: 393582 | B/O | Total

Drawing Number

Title: TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#3
SHEET 4 OF 4

PLAN VIEW

SCALE 1"=100'



Pump Station #4 Collection Conduits East Side

STA 0+00 TO 12+04

PROFILE VIEW

H. SCALE 1"=100'

V. SCALE 1"=10'

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Rev	Date	Drawn	Description	Ch'k'd	App'd

Date ________

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security STD

Project Number: 393582

B/O

Total

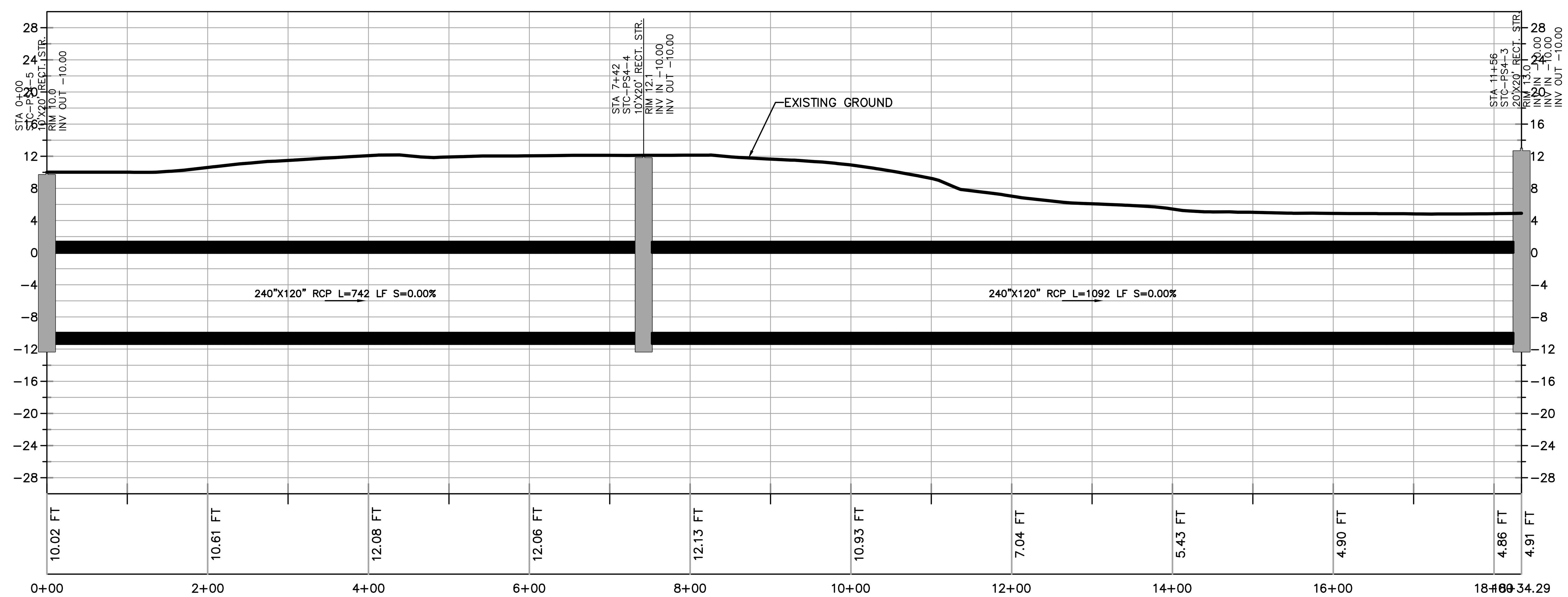
Drawing Number

Title

TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#4
SHEET 1 OF 2

PLAN VIEW

SCALE 1"=100'



Pump Station #4 Collection Conduits West Side

STA 0+00 TO 18+34

PROFILE VIEW

H. SCALE 1"=100'

V. SCALE 1"=10'

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Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security STD

Project Number 393582 | B/O | Total | Drawing Number

Title

TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#4
SHEET 2 OF 2

PLAN VIEW
SCALE 1"=100'

Pump Station #2 Collection Conduits
STA 0+00 TO 32+33

PROFILE VIEW
H. SCALE 1"=100'
V. SCALE 1"=10'

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Client: USACE

Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	

Scale at ANSI D	Status	Rev	Security
			STD

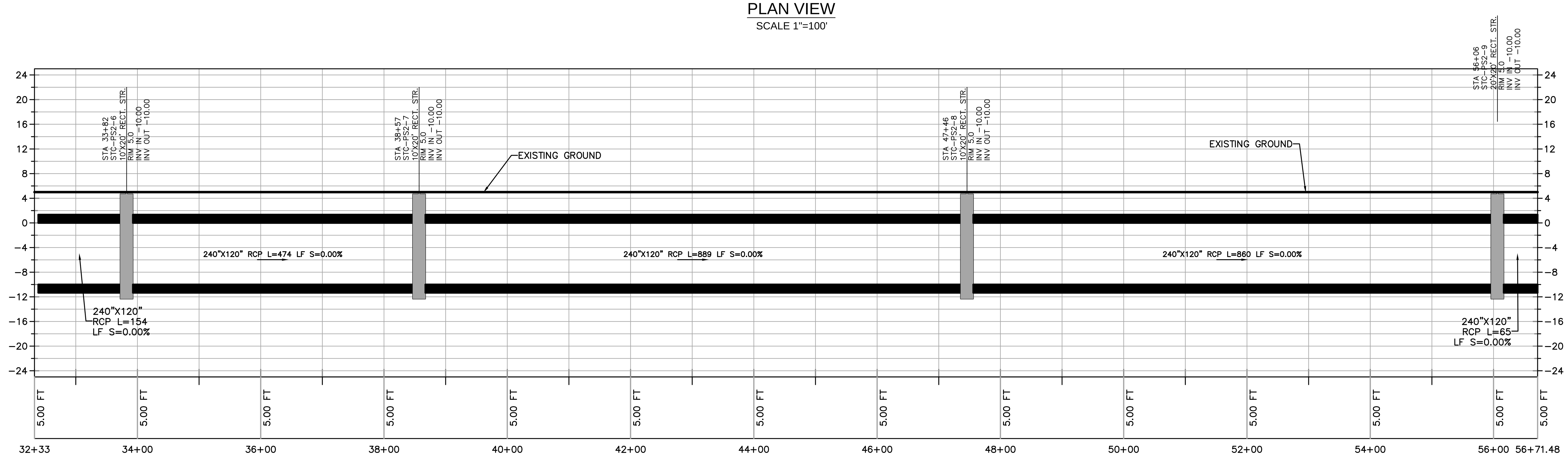
Project Number	B/O	Total
393582		

Drawing Number

Title
TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#1
SHEET 1 OF 2

PLAN VIEW

SCALE 1"=100'



Pump Station #2 Collection Conduits
STA 32+33 TO 56+71

PROFILE VIEW

H. SCALE 1"=100'
V. SCALE 1"=10'

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Client: USACE

Rev	Date	Drawn	Description	Ch'k'd	App'd

Date

Designed	ZMS	Eng check	
Drawn	ZMS	Coordination	
Dwg check		Approved	
Scale at ANSI D	Status	Rev	Security STD

Project Number: 393582
B/O
Total
Drawing Number

Title:
TEXAS COASTAL PROTECTION AND RESTORATION PROJECT
INTERIOR DRAINAGE AND HYDROLOGY
COLLECTION SYSTEM FOR PUMP STATION#2
SHEET 2 OF 2

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