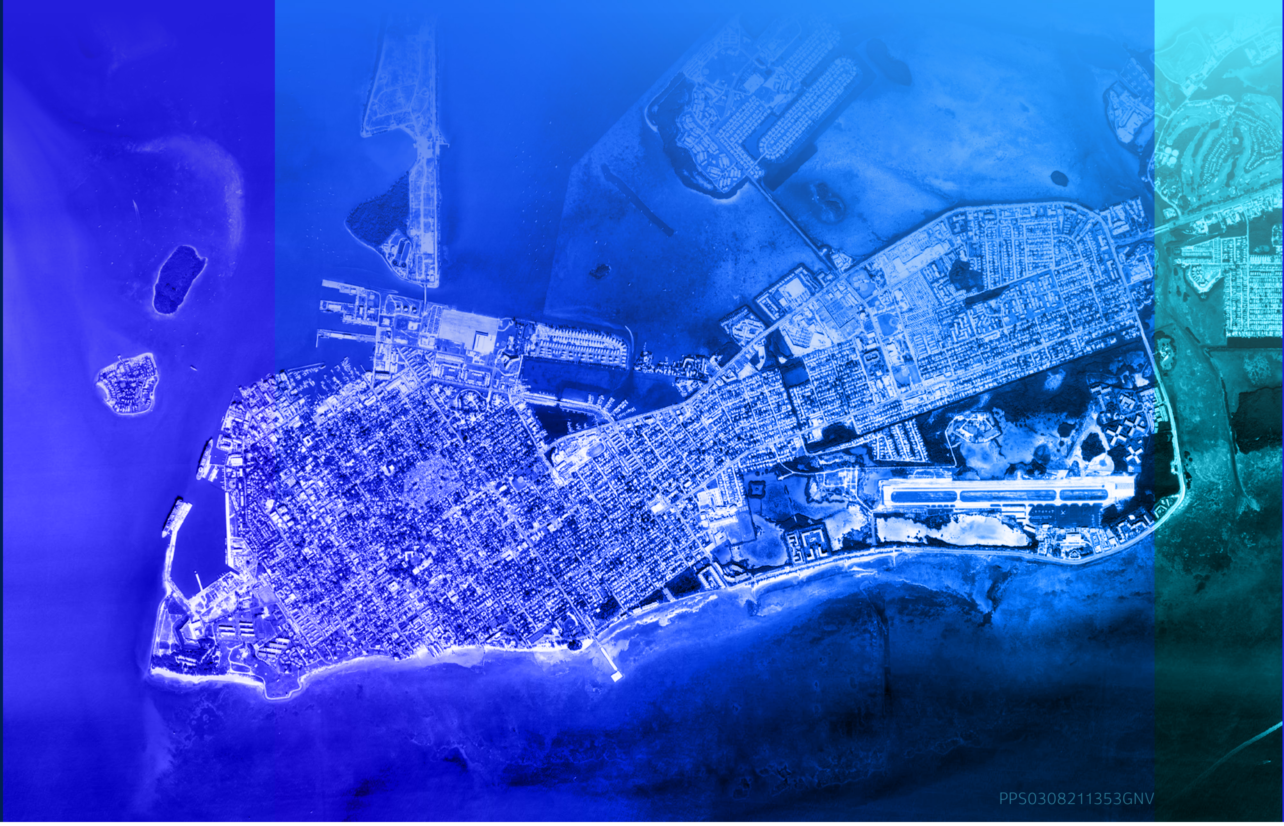
City of Key West

2024 Stormwater Master Plan Update

Phase 2
Final Report

February 2024



Jacobs

2024 Stormwater Master Plan Update

Final Report

City of Key West

Task Order 2-21
February 2024

Mitchell Lee Griffin

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Executive Summary

The City of Key West (City) occupies the island of Key West, some nearby small islands, plus the northern half of North Stock Island (north of U.S. Highway 1 [U.S. 1]). The City is only responsible for operating its own stormwater systems (not the County's, U.S. Navy's, or Florida Department of Transportation's [FDOT's]) and this report focuses solely on the City's facilities. However, the governmental agencies cooperate to provide service to the community. The goal of the Stormwater Master Plan (SWMP) is to help guide the City in developing future stormwater capital projects. The City has routinely updated its SWMP with new information approximately every 10 to 15 years. This report updates the City's 2012 SWMP (CH2M 2012). The intent of this study was to develop a SWMP that updates the City's facilities in its hydrologic and hydraulic (H&H) model, identifies and prioritizes new projects, incorporates new estimates of sea-level rise (SLR) impacts on boundary conditions (the ocean), and conceptualizes projects at selected locations to reduce flooding generated by rainfall under future conditions.

This update effort was completed during a 3-year period. The original scope of work for the City of Key West Stormwater Master Plan Update included two phases that were spread over fiscal years for funding purposes. Jacobs Engineering Group Inc. (Jacobs) initially completed a 2021 update of an Interconnected Channel and Pond Routing, Version 4 (ICPR4) H&H model, which incorporated recent projects completed since the end of 2011 (Jacobs 2021a). This new ICPR4 model was used to re-evaluate flood elevations under similar boundary conditions as the 2012 SWMP. Inundation levels were reviewed and level-of-service metrics related to length of roads flooded during a 10-year event and parcels experiencing flooding during a 100-year event were identified. The City then conducted a review of its SLR policy and established desired future ocean boundary levels corresponding to service life. This SLR policy is only guidance at this time. The Phase 2 work then proceeded in 2022 to develop projects at several locations that would meet future SLR boundary conditions. In 2023, the City asked Jacobs to add another study area and complete the SWMP.

Specifically, this 2024 Update was to identify the types of projects needed to address potential SLR in the next 30 years in selected study areas. The results of this SWMP help the City to recalibrate its expectations for the type of projects needed to maintain resiliency to climate change. With a rising ocean level, future projects are going to need pump assistance to remove runoff from the flat, low landscape on Key West. This report documents these efforts and findings.

The selected project areas were examined for near-term gravity-based flows and then with pump stations. When SLR reaches an elevation of 2.7 feet North American Vertical Datum of 1988, most outfalls serving low topography will need pump assistance to drain to the higher boundary condition. Pump stations were added in big increments, such as 50, 80, 100, and 150 cubic feet per second sizes. These future pumps stations are much larger than existing pump stations on Key West. Pipes were sized to convey these flows. Two conceptual project cost opinions were provided, with and without the pump stations. These projects show the scale of improvements needed in the selected study areas. The sizing of the pipes and pumps must be better defined during the future design projects. There are additional details and potential utility conflicts that must be considered when designing the projects. Every project area identified requires substantial resources to reduce flood stages and address future SLR.

The report also provides some additional considerations that could be looked at on a larger scale, called regional projects, to improve resiliency. These considerations include linking more areas to large pump stations, alternative design for creating higher staged water over gravity wells, tidal barriers, and oversized culverts to add storage.

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Acronyms and Abbreviations

2012 Update	Stormwater Master Plan Update, 2012
2-D	two-dimensional
AACE	Association for the Advancement of Cost Engineering
aka	also known as
BMP	best management practice
cfs	cubic feet per second
CH2M	CH2M HILL Engineers, Inc. (now Jacobs Engineering Group Inc.)
City	City of Key West
Compact	Southeast Florida Regional Climate Change Compact
DEM	digital elevation model
ENR	Engineer News Record
ERCP	elliptical reinforced concrete pipe
FDOT	Florida Department of Transportation
FEMA	Federal Emergency Management Agency
GIS	geographic information system
GPS	global positioning system
H&H	hydrologic and hydraulic
ICPR, ICPR3	Interconnected Pond Routing, version 3
ICPR4	Interconnected Channel and Pond Routing, version 4
IPCC	Intergovernmental Panel on Climate Change
Jacobs	Jacobs Engineering Group Inc.
KCA	Kisinger Campo & Associates
LF	linear feet
LiDAR	light detection and ranging (elevation remote sensing)
LOS	level of service
MHHW	mean higher-high water
MHW	mean high water
MS4	municipal separate storm sewer system
MSL	mean sea level
NAVD 88	North American Vertical Datum of 1988
NGS	National Geodetic Survey
NGVD 29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
O&M	operations and maintenance
Perez	Perez Engineering & Development, Inc.
RCP	reinforced concrete pipe
SCS	Soil Conservation Service
SFWMD	South Florida Water Management District

SLR	sea-level rise
SMU	stormwater management utility
SWMP	Stormwater Master Plan
TM	technical memorandum
U.S. 1	U.S. Highway 1
USDA	United States Department of Agriculture

1 Introduction

The stormwater system of the City of Key West, Florida (City) consists of approximately 63 permitted outfalls and associated stormwater collection systems, 54 vertical exfiltration drains, 7 pressurized wells (at four locations), approximately 109 stormwater gravity recharge wells[1], one stormwater pump station to an internal discharge location, and associated collection and treatment systems. Most of the collection system is limited to drainage around intersections. Many residential streets have inlets around the intersection corners and there are drainage wells connected to them. Only a few streets that Key West owns and operates have longer stormwater sewers (Duval Street area, for example). There are other small facilities that include open-bottom catch basins and swales that assist in allowing ponded water to infiltrate into the porous soils on the island.

The City occupies the island of Key West, some nearby small islands, plus the northern half of North Stock Island (north of U.S. Highway 1 [U.S. 1]). Both Fleming Key and Sigsbee Park are part of Naval Air Station Key West and are inaccessible to the public; the U.S. Navy operates other properties on the island as well. Sunset Key (near Mallory Square) is residential and is part of the City but is physically isolated except for sanitary sewer service. The City is only responsible for operating its own stormwater systems (not the County's, U.S. Navy's, or Florida Department of Transportation's [FDOT's]) and this report focuses solely on the City's facilities. Key West operates its municipal separate storm sewer system (MS4) under a federal permit, called an MS4 National Pollutant Discharge Elimination System permit. The City's operations are partially funded by a stormwater management utility (SMU).

Properties in the City of Key West contribute to the SMU, but not all properties are included in the MS4. By nuance of the Clean Water Act regulations, properties that discharge directly to waters of the United States, which includes some of the canals, tidal wetlands, and coastal waters, are not included in the MS4, per se. However, Florida judiciary rulings has recognized that SMUs serve the greater good of all of the community by maintaining access and services and are consistent with legislative intent. Consequently, all property in the City of Key West must abide by the SMU ordinance.

The goal of the Stormwater Master Plan (SWMP) is to help guide the City in developing future stormwater capital projects. It is expected that the City will routinely update its SWMP with new information approximately every 10 years. This report is updating the City's 2012 SWMP (CH2M 2012). The objective and scope are provided in the following sections, then a general overview on the development of the City's stormwater system is provided as background information.

1.1 Project Objective

The intent of this study is to develop a SWMP that updates the City's facilities in its hydrologic and hydraulic (H&H) model, identifies and prioritizes new projects, incorporates new estimates of sea-level rise (SLR) impacts on boundary conditions (the ocean), conceptualizes projects at selected locations to reduce flooding generated by rainfall under future conditions, and provides estimated cost opinions.

[1] Vertical exfiltration drains may only be a pit or one 8-foot section. The gravity recharge wells are approximately 80 to 100 feet deep and are permitted by the Florida Department of Environmental Protection (FDEP) as Class V injection wells.

1.2 Project Scope of Work

This update effort was completed during a period of a few years. The original scope of work for the City of Key West Stormwater Master Plan Update included two phases that were spread across fiscal years for funding purposes. Jacobs Engineering Group Inc. (Jacobs) initially completed a 2021 update of an Interconnected Channel and Pond Routing (ICPR), Version 4 (ICPR4) H&H model, which incorporated recent projects completed since the end of 2011 (Jacobs 2021a). This new ICPR4 model was used to re-evaluate flood elevations under similar boundary conditions as the 2012 SWMP. Inundation levels were reviewed, and level-of-service (LOS) metrics related to length of roads flooded during a 10-year event and parcels experiencing flooding during a 100-year event were identified.

Between phases, the City developed a draft policy for SLR that was considered during Phase 2. This report consolidates the phased work completed for the project and added an additional area of study that was analyzed after the completion of Phase 2 evaluation. Table 1-1 lists a summary of the project tasks for the update, including this report for Task D.

Table 1-1. SWMP Update Phases 1 and 2 Scope of Work

Task	Summary Description	Deliverables
A	Collect data with special effort on the as-built data of recent projects, update geographic information system layers related to stormwater, and update digital topography information.	None to City. Data were fed into the model update.
B	Convert stormwater model to ICPR4, update model to reflect new projects and information (rainfall), execute existing conditions, and summarize results. One potential SLR condition was simulated to demonstrate the potential change to the results.	Technical memorandum (TM) documenting Tasks A and B. Workshop to review results, agenda, presentation materials, and meeting summary.
C	Revise updated model from Task B to incorporate a 1-foot NAVD 88 tidal boundary as well as an SLR scenario of 2.7 feet NAVD 88 and groundwater elevation increases (2 feet NAVD 88) as related to future conditions. Evaluate proposed alternatives throughout the City in selected areas and provide conceptual cost opinions for inclusion into a final report.	TM documenting Task C. Class 4 conceptual cost opinions for recommended solutions.
D	Final Report	Consolidated TM and modeling results into one report.

1.3 History of Stormwater Management in Key West

Much of the City's stormwater infrastructure was built on an as-needed basis as growth occurred. Over the years, the City realized the need to develop planning documents to assist in the prioritization of stormwater mitigation projects. A Drainage Investigation Report was prepared in 1989 (CH2M 1989) and a stormwater runoff study was completed in 1994 (KCA 1994). In 2001, the City developed its Long-Range Stormwater Utility Plan, which formed the basis of operations until 2012 (City of Key West 2001). In 2011, the City wanted to take advantage of recently obtained aerial mapping and topographic data to update its inventory of stormwater infrastructure and to develop the 2012 Stormwater Master Plan (CH2M 2012). The City constructed several stormwater projects identified in its 2012 Master Plan. Phase 1 was to update the simulation models and to begin preparation of a new Stormwater Master Plan (2024 Update) so new projects can be identified for the next 7 to 10 years.

The City drainage systems are a combination of infrastructure designed to standards at the time they were constructed, but some older systems were nonstandard (that is, too small) when constructed. Many of these nonstandard systems appear to have been built by developers. Other nonstandard systems appear to have been built by City staff with whatever pipe and materials were on hand at the time of construction. These older (prior to 2000) outfall collection systems were not designed with pollution control as required by today's standards. There are several artificial and natural drainage systems that also serve the City.

Prior to the 1980s, stormwater gravity recharge wells were not as prevalent as they are today. The oldest operational City well located on Margaret Street between Virginia and Catherine Streets was constructed prior to the 1970s. The history of this well is unclear. A total of 12 wells were built as part of the City development of Mallory Square, Key West Bight parking lots, police and fire facility parking lot, and Southernmost Point.

In 1989, the City initiated its comprehensive planning efforts. CH2M HILL Engineers, Inc. (CH2M) was tasked to begin the process of identifying drainage structures through field investigation, because plans did not exist in City records for much of the drainage system. The Drainage Investigation Report was completed in 1989.

As required by the City's Comprehensive Plan, Kisinger Campo & Associates (KCA) performed a 1994 stormwater runoff study that identified and mapped flood problems (KCA 1994). The report included aerial mapping using surveyed ground controls. Some surveying of stormwater facilities was included in the scope of work. Eight flood areas (several blocks large in some cases) were identified and ranked by severity. The number of structures and cost to address these problems were estimated. This report highlighted the state of the stormwater system and noted many deficiencies, such as clogged inlets, too few and poorly placed inlets, collapsed outfalls, and other similar problems common with an aged system. The 1994 KCA study recommended future work to include modeling and design as funds became available. A total of 20 wells were built by the City Engineering Department in the flood zones identified in the 1994 KCA report. The City also created the stormwater program in the Utilities Department in part resulting from the recommendations of that report.

The Utilities Department began a process of developing an Inventory of Its system (both sanitary and stormwater) and cleaning and repairing its sewers. In general, sanitary repairs were implemented at a higher priority because of health concerns. Regardless, progress also was made in improving the storm sewer system. The City created a Long-Range Stormwater Utility Plan in 2001 that identified 15 flood zones, which were principally located in low areas (City of Key West 2001). The plan further documented existing systems and identified capital projects and funding requirements. The City stormwater plan incorporated policies set out in a City-generated Water Quality Improvement white paper that was the basis for the policy related to diverting water from outfalls primarily by shallow recharge wells (City of Key West 2010a). One well was built in a flood zone identified in the 2001 Long Range Plan but funding limitations kept the ambitious plan from being implemented.

Based on changes to state law and rules, the City implemented a stormwater utility to fund its stormwater program in 2003. This utility allowed the City to implement more projects than was previously possible. The City's Utilities Department led the installation of additional wells to address standing water problems not identified in the KCA Report or the 2001 Long Range Plan. In 2006, Perez Engineering & Development, Inc. (Perez) and Parsons prepared a Draft Design Memorandum for the City that updated the mapping and the City computer simulation model of the drainage system (Perez and Parsons 2006). This report helped identify additional locations where recharge wells may be located. This 2006 work provided the City with Adobe and AutoCAD maps of its stormwater system. Many of the existing inlets were surveyed to obtain elevations, and the main island was simulated in the ICPR computer model. This 2006 ICPR model was referred to as the City's stormwater system model in the 2012 report. The 2012 Stormwater Master Plan project updated the ICPR model to then current version 3 and included new information available from the geographic information system (GIS) inventory (CH2M 2012). The boundary condition and gravity wells rating curves were updated using the current information. However, the overall sub-basins and general layout of the model were still based on the 2006 work. This 2024 Update reviewed some of the sub-basins where new information was developed and has modified this City stormwater system model further.

A total of 49 stormwater gravity recharge wells were installed by the City's Engineering Department (not including the original Margaret Street well). The Utilities Department has constructed approximately 66 additional stormwater gravity wells and 7 stormwater pump-assisted injection wells (2 at Simonton/Front Streets, 2 at White/Casa Marina Court, 1 at Patricia/Ashby Streets, and 2 at Ashby/Catherine Streets [aka George Street]). These pump-assisted wells also are referred to as either pressurized or pressure wells, as opposed to the gravity-fed recharge wells (gravity wells for short). In addition, the Utilities Department restored hydraulic conveyance capacity to seven critical drainage flow ways (canals/ditches) and provided for the associated environmental mitigation. These flow ways directly serve more than 14 essential stormwater collection system outfalls.

Additional work since 2012 included implementing recommendations in the report or subsequent addendums. New inlets and pipes associated with street improvements around Duval, Whitehead, Front, and Caroline Streets included installation of six new gravity wells as part of the East Front Street project (2012 to 2014), and a storm sewer system for the Kamien neighborhood (aka Patricia and Ashby project) was completed in 2021. A new pump station near Venetia and Dennis Streets that leads to an internal outfall by the salt marsh around the airport was completed in 2020. These projects were either identified in the 2012 Stormwater Master Plan (Kamien) or were included as opportunities associated with street improvements (Caroline Street). The City also has started to abandon some of the gravity wells located in low areas where the new projects can perform better. One well was abandoned at Dennis Street and 11 wells were abandoned after the Kamien storm sewer project was finished. These recent projects are discussed as part of the model updates in Section 3.3.1.

1.4 Geodetic Datums on Key West

The National Geodetic Survey (NGS) periodically alters the baseline used to measure vertical elevations. All elevations used in the 2012 and in this Stormwater Master Plan are expressed in North American Vertical Datum of 1988 (NAVD 88). Older works (and some new information) used the National Geodetic Vertical Datum of 1929 (NGVD 29). The conversion between the two is 0 foot NAVD 88 is 1.345 feet NGVD 29, but it can vary slightly from one end of the island to another. This conversion factor is average for Key West island. The NGS recently announced that it will incorporate another change to the baseline in the next few years. It is important to understand the vertical reference datum when discussing elevations, including SLR. Section 2.1.4 describes this in more detail.

2 Phase 1 Model and Mapping Updates

Phase 1 focused on updating the City's stormwater H&H model to the latest version 4 of the ICPR computer model (ICPR4) and adding new projects. The intent of the work was as follows:

- Collect existing stormwater drainage reports, record drawings of stormwater infrastructure completed since 2011, and update other available data.
- Update the City's existing stormwater drainage model with these newly compiled data. This update included converting the ICPR model from version 3 (ICPR3) to version 4 (ICPR4), which is a major change to the software.
- Execute existing conditions simulations, map flood conditions, and develop a list of priority areas subject to flooding.

The update process began with collecting available data on changes to the storm system. Then it proceeded to conduct new simulation and mapping of existing conditions.

2.1 General Conditions

The island of Key West is located approximately 130 miles southwest of Miami at the end of U.S. Highway 1 (Overseas Highway). The City of Key West consists of the main island, some surrounding smaller islands/keys, and the northern section of Stock Island located to the east of the main island. The entire Florida Keys, including Key West, are inside the Florida Keys National Marine Sanctuary boundary. The City consists of approximately 5.9 square miles of land, but the U.S. Navy occupies a large portion of the area. The main island is approximately 3.5 miles long and 1 mile wide. Key West is the county seat of Monroe County. The county airport also occupies approximately 250 acres of land created primarily on fill in a salt marsh on the southeast side of the island (a former Naval Air Station). North Stock Island is delineated from the southern island (county land) by U.S. 1 on the southern boundary. Figure 2-1 shows the general land use on the island. City and county offices and U.S. Navy and county lands are shown as government use; schools, hospital, fire stations, and similar are shown as institutional use. These uses were derived from the Monroe County property appraiser database. The main island of Key West is primarily residential and commercial, excluding the naval bases. North Stock Island contains Florida Keys Community College, a closed landfill, hospital, elementary school, golf course with residences, botanical gardens, miscellaneous smaller businesses, and a county jail.

2.1.1 Topography

The City was initially developed on the higher land on the western portion of the main island, and this area is known as Old Town (generally west of 1st Street). The elevations are higher just east of Duval Street at nearly elevation 15 feet NAVD 88. Old Town's landscape slopes down toward the Gulf of Mexico to the north and the Atlantic Ocean to the south. East of 1st Street is called New Town and is relatively flat. Figure 2-2 shows the general topography based on recent digital elevation model (DEM) data. This figure also includes the sub-basins used in the existing conditions model.

In 2008, the Florida Division of Emergency Management conducted a coastal mapping project that collected high-resolution aerial photographs and topographic (elevation) data, which were used in the 2012 Stormwater Master Plan. A newer published DEM topography was downloaded from the National Oceanic and Atmospheric Administration (NOAA) that was produced for the Coastal Management's Sea Level Rise and Coastal Flooding Impacts Viewer (NOAA 2020). NOAA created the DEM based on available light detection and ranging (LiDAR) data available at the time of DEM creation. These DEMs are the datasets used by NOAA to visualize the impacts of inundation resulting from SLR along the coastal United States and its territories. In general, this alternative data source should not vary from the 2012 data, but processing of the LiDAR data may produce slightly different results. It was downloaded and used in this update to be consistent with other regional interpretations of potential coastal effects. The field survey data of stormwater facilities were the priority source of elevation information, and the DEM was for general modeling use only.

2.1.2 Geology

Key West is generally a low barrier island consisting of a layer of sandy or marly soil, typically 3 to 5 feet deep, on top of an oolitic limestone base. Freshwater seeping into the ground forms a thin layer of less dense- water and mixes with saltier groundwater over tidal cycles. The limestone is porous and because of cavities or cracks, it can be often very transmissive for groundwater. Because of this porous characteristic and because there are no potable deep groundwater sources in the City, shallow recharge wells are used for stormwater control. Figure 2-3 shows the U.S. Department of Agriculture (USDA) Soil Survey for the island. Most of the island is listed as urban, which is a general term the USDA uses for developed land, and no published soil data exist. Most soil data are available from soil borings conducted during construction projects or City staff's general knowledge.

Experience has shown that the depth to rock varies greatly from one location to another. The groundwater table fluctuates with the tides because of the porosity of the rock. The elevation of the groundwater table is normally approximately 0.5 foot to 1.5 feet above the tide levels depending on proximity to the coast.

2.1.3 Climate Characteristics

Because of the proximity of the Gulf Stream to the Straits of Florida (approximately 12 miles) and the tempering effects of the Gulf of Mexico to the west and north, Key West has a notably mild, tropical maritime- climate where the average temperatures during the winter are only approximately 15 degrees Fahrenheit lower than in summer. Humidity remains relatively high during the entire year. There is no known record of frost, ice, sleet, or snow in Key West. Precipitation is characterized by dry and wet seasons. The period of December through April receives slightly less than 25 percent of the annual rainfall. This rainfall usually occurs in advance of cold fronts in a few heavy showers, or occasionally five to eight light showers per month. June through October is normally the wet season, receiving approximately 53 percent of the yearly rainfall total in the form of numerous showers and thunderstorms. Early morning is the most likely time for precipitation (Key West Chamber of Commerce 2021). Direct hurricane strikes are not common, but the City has experienced several severe windstorm flooding events (Tropical Storm Fay and Hurricanes Wilma and Irma are notable recent windstorms).[2]

Table 2-1 includes a summary of monthly totals for precipitation and temperature. Included in this summary are the number of days with precipitation totals that exceed 0.1 and 1.0 inch. The 0.1 inch threshold is important because this value often is used to distinguish storms that may cause enough runoff to have a measurable effect, which are approximately 62 storms per year at the City. The larger storms are much fewer, approximately 11 storms per year. Of note in this NOAA dataset is the highest daily storm of 22.75 inches that occurred in November 1980. As shown in Table 2-1, storms with rainfall totals of more than 4 inches can occur almost any time of the year.

[2] Refer to http://www.hurricanecity.com/city/keywest.htm for more information on hurricanes.

Figure 2-1. General Land Use on Key West

Figure 2-2. General Topography based on Recent Digital Elevation Model Data

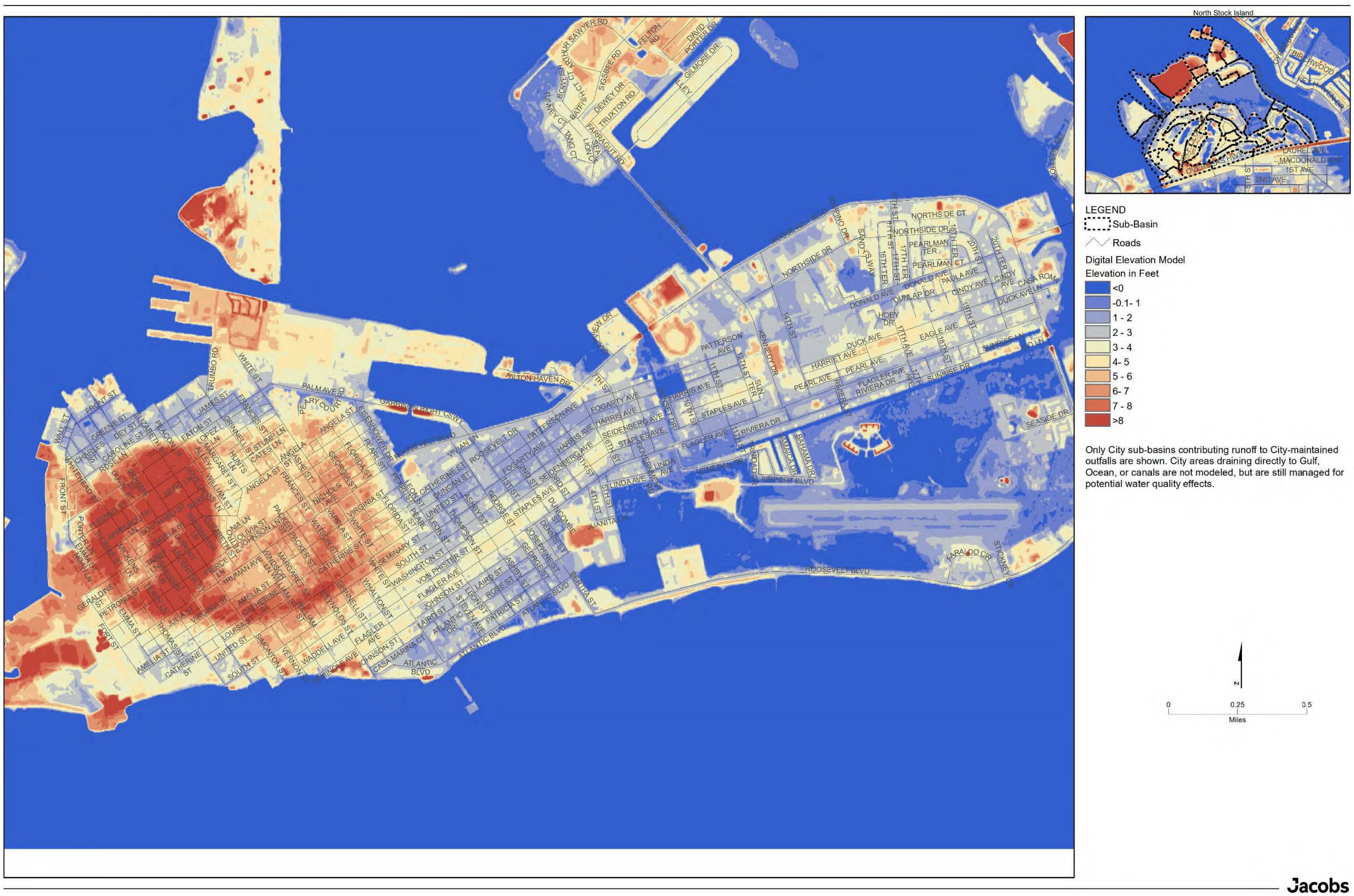


Figure 2-3. USDA Soil Survey Results

Table 2-1. Summary of Long-Term Climate Data for Key West, Florida

Month	Temperature[a]			Precipitation				
	Daily Max	Daily Min	Mean	Mean[b]	Median[b]	Highest Daily[b]	Mean No. of Days[c] ≥0.1 inch	Mean No. of Days[c] ≥1 inch
Jan	74.3	64.2	69.3	2.00	0.98	6.42	3.1	0.4
Feb	76.0	66.0	71.0	1.54	1.22	4.34	2.7	0.4
Mar	78.2	68.3	73.2	1.85	1.48	5.26	3.5	0.5
Apr	81.3	71.6	76.4	2.15	1.74	6.19	2.8	0.6
May	85.0	75.7	80.3	3.27	2.68	4.14	3.9	0.9
Jun	87.8	78.8	83.3	4.07	3.47	5.14	6.3	1.2
Jul	89.3	79.8	84.5	3.74	3.25	4.25	6.5	0.9
Aug	89.4	79.6	84.5	5.30	4.38	9.66	8.9	1.5
Sep	87.9	78.5	83.2	6.54	6.36	9.37	10.5	1.7
Oct	84.5	76.0	80.2	4.92	3.32	7.30	7.0	1.3
Nov	79.9	71.7	75.8	2.81	1.51	22.75	3.3	0.7
Dec	76.0	66.9	71.4	2.19	1.45	6.66	3.2	0.6
Annual	82.9	73.2	78.1	40.40	39.33	22.75	61.7	10.7

Source: NOAA (2021)

[a] Monthly summary of temperature from 1981 to 2010.
[b] Daily precipitation from 1980 through 2020.
[c] Monthly summary of events from 1981 to 2010.

≥ = greater than or equal to

2.1.3.1 Design Storms

An important rainfall data input is the storm characteristics used to assess and design infrastructure. Design storms are expressed in terms of a return period, which is an expression of probability (= 1/return period). For example, a 10-year storm means that there is a 1 in 10 chance that a storm at least as large as that one would occur in any given year. The Federal Emergency Management Agency (FEMA) often uses the 100-year storm (1 percent chance of occurrence per year) as a threshold for determining flooding potential. Building codes normally require the minimum first-floor elevation to be above the 100-year flood elevation. However, extensive flooding can occur with much smaller storms, including the 2-year storm (50 percent chance of occurrence).

The design storms used in the evaluation are shown in Table 2-2. In 2012, the rainfall volumes are based on standard literature values available from either the South Florida Water Management District (SFWMD) or FDOT. Since 2012, NOAA has published updates to precipitation analysis, commonly referred to as Atlas 14 (Perica et al. 2013). The 2024 Update used the most recent available data, which are shown in Table 2-2. Some projections expect that rainfall volumes for design storms are going to increase with climate change because higher atmospheric temperatures will carry more moisture. As part of the assessments for the City's SLR policy review (Jacobs 2021b), the projected volume of future rainfall was examined by Jacobs but it was determined that future rainfall was not going to change by a statistically significant amount. The mixture of high values between Atlas 14 and SFWMD was retained for this update.

Table 2-2. Design Storms for the City of Key West

Return Period (years)	Duration (hours)	Distribution	2012 SWMP Storms (inches)	2013 Atlas 14 Volume (inches)
2	24	FLMOD	5	4.8
5	24	FLMOD	6	6.2[a]
10	24	FLMOD	7	7.6[a]
10	72	SFWMD72	10.5	9.25
25	24	FLMOD	9	9.9
25	72	SFWMD72	12[a]	11.9
100	24	FLMOD	12	14.1
100	72	SFWMD72	17[a]	16.9
500	24	Not simulated	NA	20.3
500	72	Not simulated	NA	24.2

[a] Volumes used in the 2024 Update.

Used ICPR distributions as identified above: Florida-modified Type II storm (FLMOD) or the SFWMD 72-hour distribution (SFWMD72). Atlas 14 volumes were used for this report.

NA = not applicable

The SFWMD guidance was used to establish the time distribution of rainfall intensities (hyetographs). The 500-year storms were not simulated but are listed because some new guidance is now referring to this storm for critical infrastructure design (Southeast Florida Regional Climate Compact 2020).

2.2 Previous Reports

There were two reports that summarized most of the data used from historical studies prior to the 2012 Stormwater Master Plan. The first report is a historical summary of the City's stormwater program compiled by the City in 2010 (City of Key West 2010). The report details the development of the City's program and includes several design guidelines that the City is currently using. The second reference is the 2006 Perez and Parsons Design Memorandum that provides the updated City ICPR model and a brief update on the recent projects (Perez and Parsons 2006). All new stormwater facility data prior to 2011 were incorporated into the ICPR version 3 model used in the 2012 Stormwater Master Plan.

This report used the 2012 Stormwater Master Plan as a starting point and then added or updated data sources as available. Additional infrastructure that was built since the 2012 report was based on plans provided by the City. No new survey was conducted as part of the 2024 Update. Changes that were incorporated into the stormwater model are described in Section 2.3.4.

2.2.1 Datum and Tidal Levels

Elevations on a landscape are set relative to long-term elevations of the ocean and a network of fixed benchmarks. The NGS maintains this network and updated the historic standard established in 1929 with a new standard referred to as the 1988 datum. In practical terms, the landscape has not moved, but the yardstick used to measure the elevation has been shifted. The SFWMD and most municipalities have traditionally required the NGVD 29 for surveying and expressing elevations. However, most municipalities are in the process of switching to NAVD 88. Available literature or survey are presented in one or the other reference datum. Except for recent work (post-2010 or so), elevations typically are expressed in NGVD 29. The conversion between NAVD 88 and NGVD 29 in Key West is to subtract 1.345 feet, so the reported NAVD 88 elevation would be lower for the same location. This conversion may vary slightly from one side of the City to the other, but this difference would be slight and of little consequence to normal public works facilities. The conversion was computed using the NGS Coordinate Conversion and Transformation Tool program at latitude 24°33′26″N and longitude 81°47′14″W (NOAA 2021).

A main consequence of the datum conversion is a restatement of the sea level elevations surrounding Key West. The main NOAA tide gauge at Key West (ID: 8724580) is located in the boat basin on the west side of the main island. The updated tide levels are presented in Table 2-3 and are based on NOAA tide data from 1983 through 2001. This is the same epoch used in the 2012 Master Plan. As shown in Table 2-3, mean sea level used to be near 0.2 in the NGVD 29 reference datum, but is now expressed close to 1.5 under the new NAVD 88 datum. Similarly, stormwater evaluations often are conducted under mean high-water conditions that used to be near elevation 1.1 NGVD 29 but are now close to 0.2 NAVD 88.

For purposes of this and the 2012 modeling, the boundary condition at the ocean was set at elevation 0 NAVD 88, closer to the current NOAA-listed mean higher-high water (MHHW). MHHW represents the average of the higher daily tide levels. This higher level is consistent with the City design policy (City of Key West 2010). Because of SLR, this boundary condition has changed (risen) because the epoch averaging period is now nearly 20 years old. This is explained in the next section.

Table 2-3. Tide Levels at the Key West NOAA Gauge in Different Vertical Datum

Description	Acronym	Elevation NAVD 88	Elevation NGVD 29
Mean Higher-High Water	MHHW	0.05	1.40
Mean High Water	MHW	-0.24	1.11
Mean Tide Level	MTL	-0.88	0.47
Mean Sea Level	MSL	-1.52	-0.18
Mean Lower-Low Water	MLLW	-1.76	-0.42
Mean Range of Tide	MN	1.28	1.28
Highest Astronomical Tide 10/17/1989	HAT	0.89	2.34
Highest Water Level	MAX	1.98	3.33
	MAX DATE	9/8/1965	

All elevations are in feet.

MSL = mean sea level

Based on NOAA Gauge 8724580 for Key West, accessed 2/18/2021.

2.2.2 Sea Level Rise

The change in vertical datum on land is only a shift in the values at which elevations are expressed, as though moving a ruler next to a fixed object (survey benchmarks). There is no physical landscape change associated with the conversion. However, there is a documented rise in sea level over time when compared to landward benchmarks resulting from an increase in volume (melting glaciers). The long-term SLR at Key West has historically been at approximately 2.5 millimeters per year (NOAA 2010). This average has risen since 2012 (was 2.2 millimeters per year). The Southeast Florida Regional Climate Change Compact has completed studies of SLR in the region since the previous Master Plan (Compact 2019). The 2019 report provided an analysis of the Key West gauge and compared it to observed data. Figure 2-4 is copied from this report, and it shows that the 5-year average has recently risen at a greater rate since 2010. However, there have been periods of rapid rise followed by slower rise in the data record. The Compact also reported the observed data against common projections of SLR, shown on Figure 2-5, and determined that the Intergovernmental Panel on Climate Change (IPCC) projection is closer to observations since 2000, but the NOAA intermediate high projection is within the range of the monthly sea levels too. Table 2-4 provides the predicted future sea level rises for Southeast Florida.

Figure 2-4. City of Key West Sea Level Trend from Compact 2019 Update

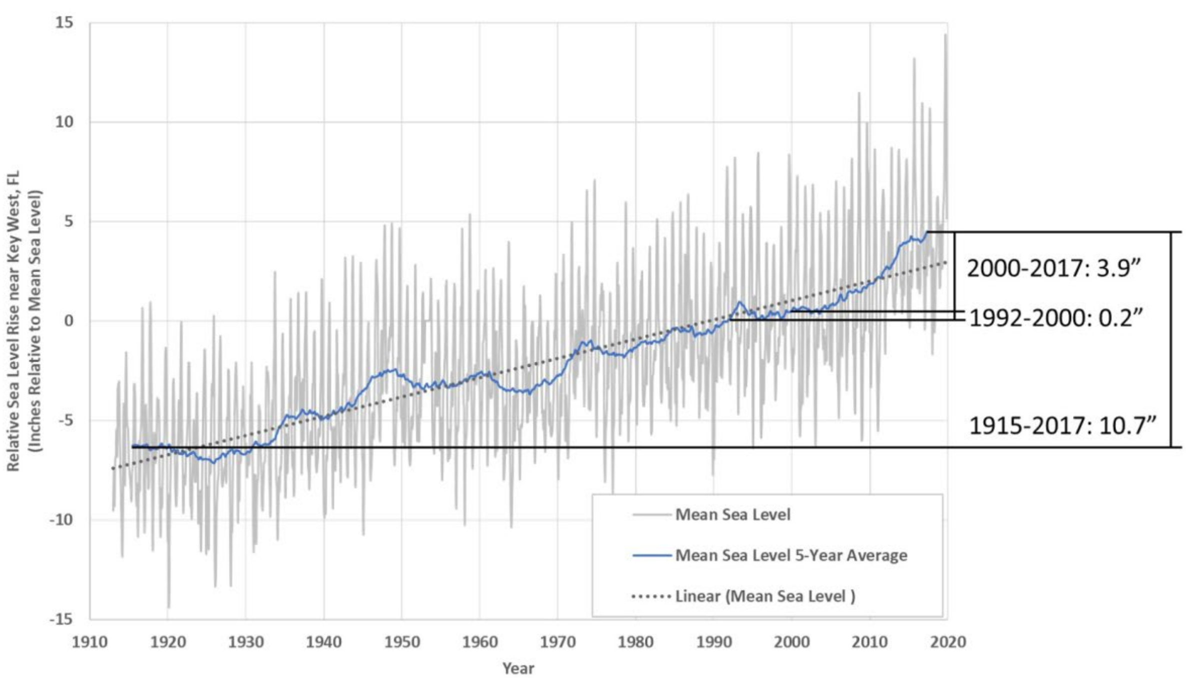


Figure 2-5. City of Key West Sea Levels Compared to Common Predictions from Compact 2019 Update

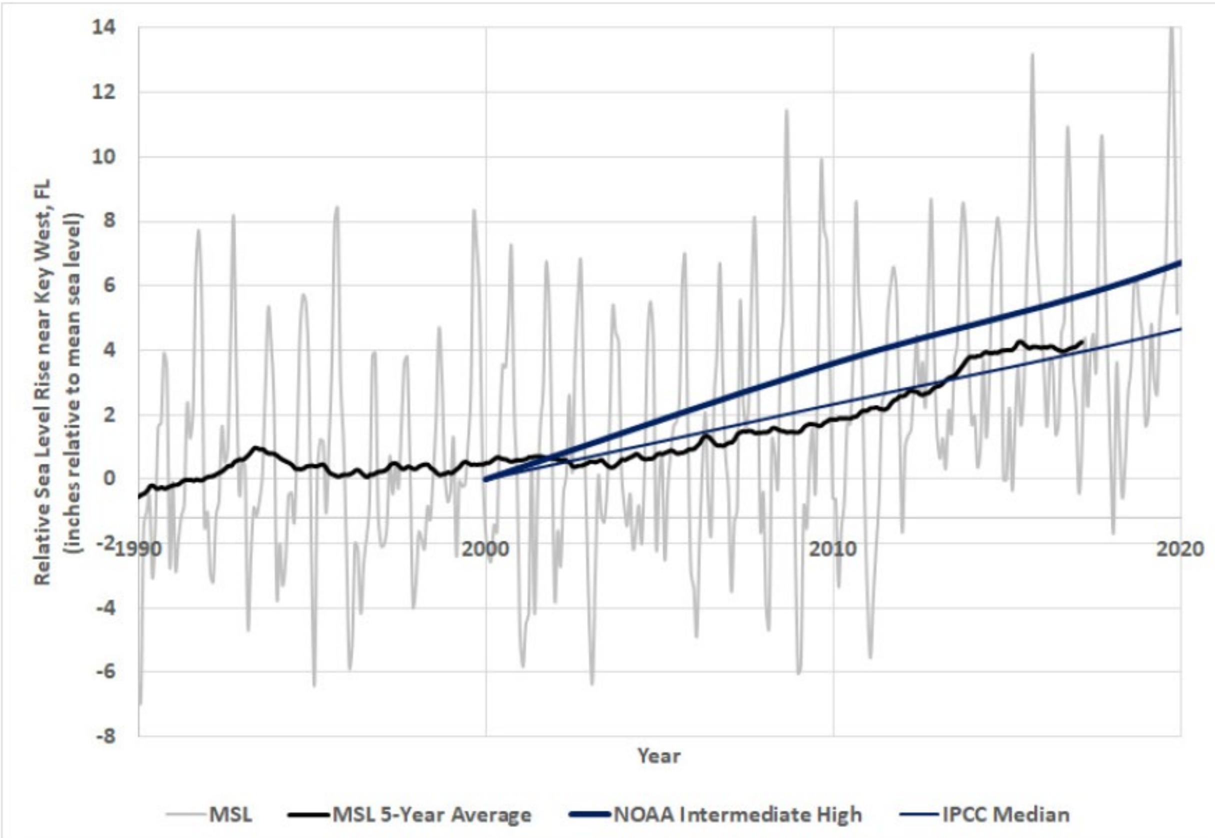


Table 2-4. Projected Sea Level Rises from Compact 2019 Update

Reference Date	IPCC Median (inches/foot)	NOAA Intermediate High Projection (inches/foot)
2000 (Compact baseline)	0	0
2012	3/0.25	3/0.25
2020 (~current)	5/0.5	8/0.75
2030 (10 years)	8/0.75	12/1.0
2040 (20 years)	10/0.8	17/1.4
2050 (30 years)	14/1.2	25/2.1
2070 (50 years)	21/1.75	40/3.3

Source: Estimated from Figure 1 in 2019 Compact report.

The currently listed data are based on observations from 1983 through 2001. This NOAA stage reference should be updated soon as there is 19 years of newer data available. As part of an assessment of SLR, Jacobs updated the tide estimates using data from 2002 through 2020 (Jacobs 2021b; Appendix B).

Table 2-5 provides a comparison of the results, which indicates that SLR has been about 0.21 foot over the past 20 years (0.126 inch/year, or 3.2 millimeters/year). The Compact projections do include an increase in the rate of change in the future and this analysis reflects the increasing trend.

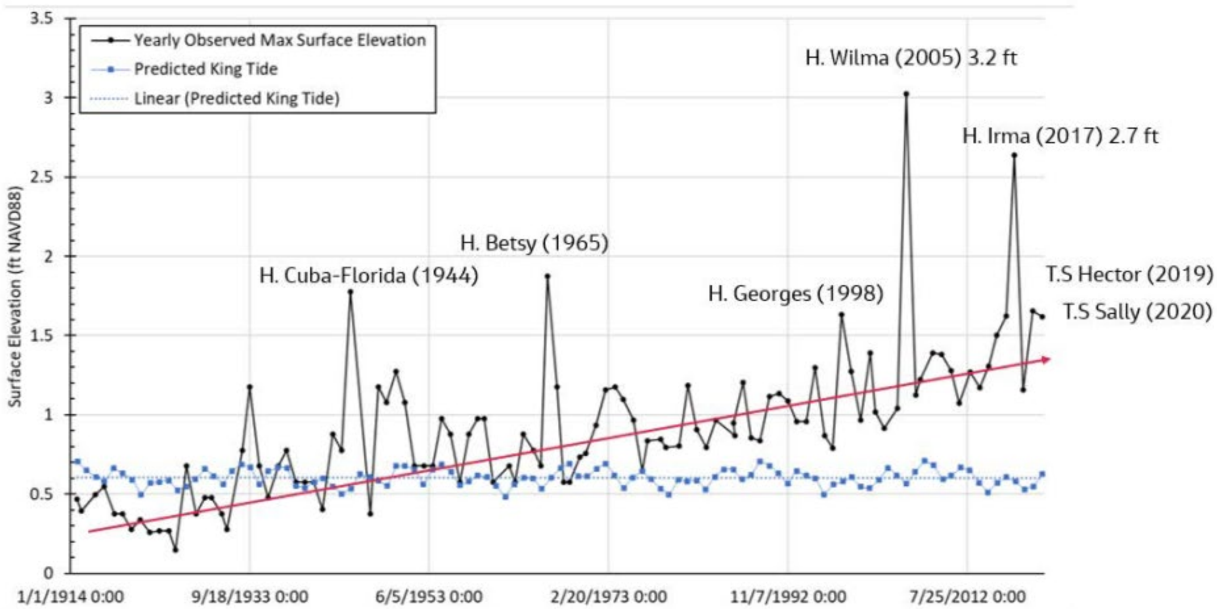
Table 2-5. Current and Potential New Tide Levels at Key West (NOAA Gauge 8724580)

Tidal Metric	Current NOAA Datum	Reanalysis (Jacobs 2021b)	Definitions and Comments
Data from:	1983-2001	2002-2020	Extreme value analysis is of the maximum tide elevation observed per year.
	All elevations are in NAVD 88		NR = Not reported
Highest Obs. Tide	3.18	--	Occurred 10/24/2005 (H. Wilma)
MHHW	0.05	0.26	Mean Higher High Water
MHW	-0.24	-0.03	Mean High Water
MSL	-0.87	-0.65	Mean Sea Level
MLW	-1.52	-1.29	Mean Low Water
MLLW	-1.76	-1.52	Mean Lower Low Water
Fall MHHW (avg)	NR	1.0*	Fall is monthly avg. MHHW Sept. to November (*regression trend fit).
HAT	0.9	NR	Highest astronomical tide.
Max. Storm Surge	--	3.2	H. Wilma 2005
			(H. Irma was 2.7, 2nd highest)
10-year Prob. Annual Exceedance	1.7**	1.9	**NOAA extrapolated the probable levels to 2018 based on older trends. Jacobs did the analysis using 107 years of data through 2020.
25-year Prob. Annual Exceedance	NR	2.2	
100-year Prob. Annual Exceedance	2.3**	3.1	Extreme value analysis of 107 years of data yields 100- and 50-year results with higher uncertainty because of length of record.

Accessed January 27, 2023: https://tidesandcurrents.noaa.gov/datums.html?id=8724580
Jacobs. 2021b. City of Key West – Sea Level Rise Policy. Prepared for City of Key West, Engineering Department. Task 1-21-ENG. August 30.

Also shown in Table 2-5 are the results of extreme value statistical analysis of the observed tides. A return period is an alternative format to express the probability of occurrence in any given year. A 25-year return period has a 1/25 = 4 percent chance of occurring in any given year. Or one could interpret this to mean that a high tidal value of at least 2.2 feet NAVD 88 has a 4 percent chance of occurring in any year. Consequently, selecting a return period is a way of applying risk into the design. Figure **2-6** provides a chart of observed high tides observed in Key West over 107 years. It is not unrelated that the 100-year return period is close to the maximum data value in the extreme value analysis. However, the main takeaway from Figure 2-6 is that the high values are trending upward (the red line is not a regression, but only there for illustration). Based on the City's resilience planning, future tidal conditions should consider the average fall season MHHW of 1 foot NAVD 88 (Jacobs 2021b). This chart also shows that the surge in Hurricane Wilma was much higher than other storms, so a value of elevation 3.2 feet NAVD 88 is currently a good high target level, but increases can be expected too.

Figure 2-6. Maximum Water Surface Elevations Observed at Key West



FEMA also predicts storm surge flooding that will be higher than the normal tidal and observed storm surges. For the coastline around Key West, the proposed 100-year storm surge estimates are approximately 11 feet NAVD 88 but reduce 2 to 3 feet as the waves move onshore. FEMA uses its projection as part of the federal flood insurance program. The Florida Building Code requires that new construction use these FEMA values for locating vulnerable electrical controls and new building finished floor elevations.

2.2.3 Reported Flooding Problem Areas

The City maintains a map showing flooding problem areas as provided on Figure 2-7. FEMA insurance claims include historical events that were not always a result of normal stormwater flooding (that is, hurricane and tropical storm claims were included). These FEMA claims are confidential data subject to the federal Privacy Act so no specific data about the affected parcels are available publicly. The City allowed Jacobs to review this map to understand where known flooding problems are located. The reported flooding problem areas generally correspond to those low spots that experience flooding already identified in previous studies and by the City through public complaints.

Figure 2-7. City Flooding Problem Areas

2.2.4 Field Data Collection

No new field data collection was conducted by Jacobs for the 2024 Update.

2.3 Hydrologic and Hydraulic Modeling of Existing Conditions

The previous 2012 Stormwater Master Plan updated the cumulative studies conducted previously by others during a 20-year span. Each new study builds on previous evaluations, and updates for new conditions, information, data, and regulatory criteria. The City contracted Perez and Parsons to build a computer model for the main island stormwater system. The 2005 City model was used as the starting point for the 2012 study. Data collected during the 2011 field inventory were used to update the City's model. In addition to field data, as-built drawings of projects were used to be sure that the revised City model was accurate. The old City model had drainage wells and some pipes and inlets in roads owned and operated by FDOT or the County, and not all of these were inventoried by the City. In these cases (mostly along Flagler, Truman, and Roosevelt Streets, both north and south drives), the former City model data were used as is. The computer modeling of the stormwater system on North Stock Island was conducted separately from the main island.

This 2024 Update similarly informed the most recent computer model of the stormwater system. After the previous Master Plan was completed, CH2M HILL (now Jacobs) conducted some additional studies that supported some design work. During these focused evaluations, the existing model was used as a starting point and often more detail was added to incorporate proposed features. Typical detail included breaking some sub-basins into smaller units to better size pipes. Thereafter, the City retained another design firm to complete some of the designs. As-built drawings of the final projects were collected, used, and incorporated into this SWMP Update. When the previous model was modified, the portions with greater detail were used to update the 2021 model. The City also implemented some of the recommendations as part of the Front Street project and added recharge wells.

This section describes how the City's 2012 computer model was updated and provides the results of the updated simulated existing conditions for the main island.

2.3.1 Overview of ICPR Program

The ICPR computer program was used to simulate the design storms in the 2005 City model and was retained for subsequent updates. This computer program is popular in Florida and is often used in designing stormwater facilities. It is a stormwater node-link model where excess stormwater is estimated to predict runoff hydrographs (flow versus time) into nodes; and links are hydraulic elements such as pipes, channels, or street overflow. A node can be a pond, manhole, or a placeholder used to connect links. The recharge wells are associated with nodes, and their impact is simulated by using stage-discharge relationships (Section 2.3.4). This simulation tool is sometimes referred to as the H&H model because it incorporates both runoff and the routing of the runoff to the boundaries in one package.

In 2012, and other work done through 2020, ICPR version 3 was used as the H&H model. The ICPR developer, Streamline Technologies Inc., issued version 4 and quit supporting its earlier ICPR version 3 in 2019, including operating systems prior to Windows 10. Consequently, a main objective of updating the City H&H model was to update it to ICPR4. This was accomplished by using import tools and then the conversion was checked against version 3 results. New facilities were added to the ICPR4 model. There are two models, one for the main island and another for Stock Island. There were no new facilities identified on Stock Island, so that version 3 model was converted to ICPR4 without other changes. This version 3 to version 4 and project update work was completed in 2021, but the SWMP was not completed until 2024.

The modeling approach previously used was retained for the 2024 Update. For simulating large storm events, hydrologic modeling entails predicting the stormwater runoff hydrograph from the sub-basins. The program was used to compute runoff using standard Soil Conservation Service (SCS) methods for the design storms described in Section 2. These SCS methods are standard practice and are accepted by the SFWMD. Unit hydrographs and rainfall distributions are defined by SFWMD criteria. Hydraulic modeling entails predicting flow rates in links and water depths at nodes in a process that is generically called routing the storm. Routing is accomplished by iterative numerical solutions to equations of physics (termed dynamic routing) that account for water staging up and backwater effects from downstream nodes. By using dynamic routing, the stormwater model can accurately compute the flows and water elevations in the entire drainage system for the flat topography in Key West. The stormwater model evaluates the capacity of the pipes, pump stations, and wells, but not the inlets. The capacity of street inlets is assumed to be nonlimiting in the computer model, which is a common assumption used in stormwater master plans. Sometimes this assumption is inaccurate, especially in older neighborhoods. This assumption requires that inlet capacity be considered independently of the modeling results during the design of new facilities.

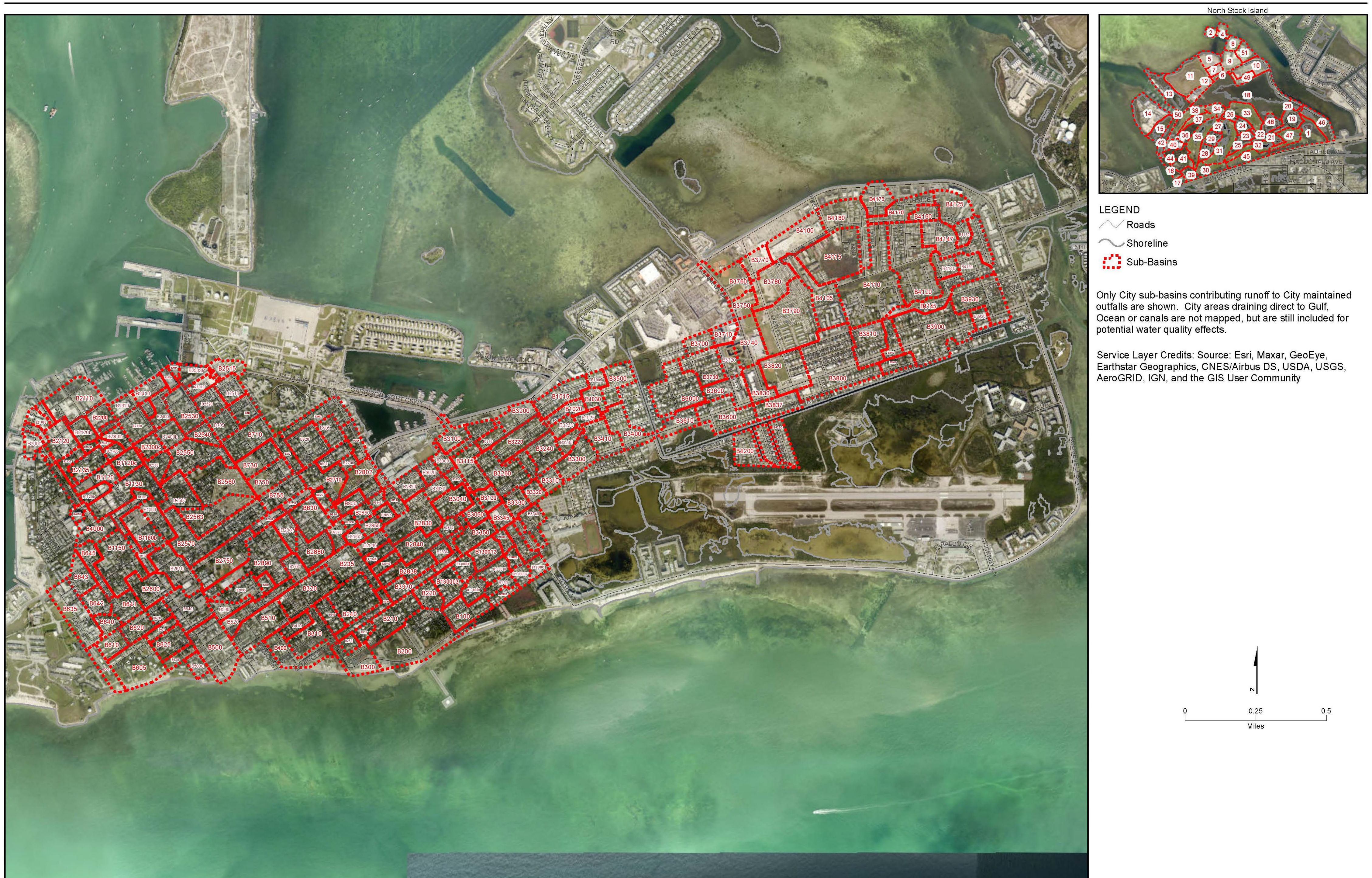
2.3.2 Sub-basin Delineation

Sub-basins are used in the hydrologic model to estimate runoff, and each sub-basin is usually associated with at least one node. The 2005 City model was used as a starting point for the sub-basin delineation. Subsequent updates (2012 and 2021) modified sub-basins to better represent the runoff to specific facilities. Although the topography was updated with more detailed mapping, the sub-basins in the City typically are defined by the pipe networks and street elevations. The blocks between major streets often form the sub-basin divides, and the 2005 work often used the sanitary sewer as-built manhole elevations near the middle of intersections as input data. The 2012 work used new global positioning system (GPS) inventory to modify the boundaries or split basins in a few locations to capture drainage features, especially for the newer facilities. This 2024 Update started with the 2012 GIS layers and incorporated the modeling studies completed after the 2012 Update, including new sub-basins. Some further adjustments were made near the Ferry Terminal area to better define the contributing areas to each outfall there. Figure 2-8 illustrates the revised sub-basins for the new City ICPR4 model.

2.3.3 Model Setup

The City’s 2005 model was reconstituted to form the basis of the 2012 ICPR3 model. A large change to the City’s model in 2012 was to change all of the input data into NAVD 88. This was done by subtracting 1.345 feet from every elevation data point in the model. The 2012 Update entered projects implemented since 2005, including the White Street pump-assisted wells and approximately 31 new intersections with gravity wells. Because the George Street pump-assisted well system was still being designed, it was not included in the 2012 existing conditions model. This facility is now updated into the 2021 H&H existing conditions model.

Figure 2-8. Revised Sub-basins for the New City ICPR4 Model



Another 2012 decision was to construct one model for the entire main island and a separate model for Stock Island. The 2005 model was only for the main island. Sub-basins were assigned to “groups” in ICPR that generally corresponded to ocean outfalls. Also, the same node and pipe naming scheme from the 2005 City model was used in the 2012 model. The detailed studies sometimes split a larger sub-basin into smaller units. New sub-basin names often were derivatives of the original name. For continuity between studies, the same names were retained for the 2021 model.

The following steps also were completed to develop an accurate update to the City’s stormwater model:

- Pipe sizes and pipe connectivity were checked against the 2011 GPS field data. If a difference in pipe sizes existed between the field data and former City model, the pipe size and connectivity provided in the field data were used. New as-built data were used to update pipe and inverts.
- Gravity recharge wells were already in the 2012 City model, and it included proposed wells that were turned off (no flow). However, some of the new projects were in other locations and some of the proposed wells were not constructed. The locations of the existing gravity recharge wells were identified from the 2011 field data and as-built drawings. A gravity recharge well is represented in the model as a node with rating curve associated with it. A typical rating curve was used for all the gravity wells (discussed further in Section 2.3.4).
- The GIS topographic data were used for elevation area for each basin. The 2012 Update used the DEM to intersect the sub-basin boundary to produce the stage-area table for each sub-basin and then data were entered into the model. This was conducted for the new sub-basins configured for the 2024 Update.

2.3.4 Specific Updates to the 2021 H&H Model

Different types of updated information were provided by the City or were available from work previously performed by Jacobs. When a study was performed by Jacobs, the project-specific model was available. The as-built survey or design plans were used to verify the final constructed facilities. There were only a few projects (since the 2012 update) to incorporate into the model, as described in the following subsections.

2.3.4.1 Patricia and Ashby Stormwater Improvements (2020)

This project included installing new inlets and stormwater pipes throughout the Kamien neighborhood and connecting them to the pressurized well facility located near the intersection of Patricia and Ashby Streets. The project was completed in July 2021. The design drawings were used to enter new pipes and inlets. The neighborhood was acting hydraulically like a bowl and was represented as one sub-basin in previous models. The design included breaking this one basin into 14 smaller sub-basins to align with the new pipe network. The project model was incorporated into the 2021 H&H model.

An add-on to this project included plugging 11 gravity wells in the neighborhood after the pipes were installed. This change was not included in the 2021 existing conditions model because the wells were not plugged yet. Because of the low landscape, the gravity wells had negligible impact on peak flood stages. These wells are now plugged and are turned off in the future conditions model.

2.3.4.2 Simonton Outfall (2016)

Emergency outfalls were added to the pressurized wells at Patricia and Ashby Streets, George Street, and Simonton Street. As part of the design of the Simonton outfall, the contributing areas along Simonton and

Front Streets were updated to reflect the final design. Sub-basin boundaries were modified in the contributing area.

The emergency outfalls were permitted for operation during severe tropical weather to protect the pressurized wells from excess sand being pumped into the wells. The permit allows use of the outfalls for these pump stations to protect property from flooding. The capacity of these outfalls is controlled by the pump capacity. The four pressurized well systems with outfalls were assumed to have their emergency outfalls turned off during the design storms. But if they were on, the pump flow rate would remain the same.

2.3.4.3 George Street Pump Station (2014)

CH2M was beginning a design of a new pressurized well system starting about the same time that the 2012 Master Plan was completed. This facility was installed, and the modeling conducted to support the design was used to upgrade the 2012 model for future conditions. For the 2021 model, the emergency outfall was included and as-built data were used to adjust the designed features into the existing conditions model.

2.3.4.4 Front Street Project (2015)

The City upgraded several outstanding items in a design project by Perez. The improvements were derived from the 2012 Stormwater Master Plan and an amendment. This project included routing the intersections around the Aquarium to a new outfall location (bigger pipe) because the old outfall failed. This project also included adding six gravity wells around Duval Street to offload runoff from the main stormwater pipe serving the commercial district. Sub-basins were split around the Wall Street and Mallory Square area, and along a couple of the streets with new gravity wells. As-built data were used to modify the ICPR4 model.

2.3.4.5 Caroline Street (2016)

As part of a street upgrade, Perez designed new stormwater inlets and upgraded an outfall between Duval and Grinnell Streets. A 12-inch outfall was replaced with a 24-inch pipe. CH2M also amended the Stormwater Master Plan and modified the stormwater model in this area to include more sub-basins to support Perez's design. The as-built drawings were used to verify the new infrastructure in the design.

2.3.4.6 Dennis Street Pump Station (2020)

The intersection near Venetia and Dennis Streets was a priority project in 2012. Black and Veatch designed a pump station to discharge to an indirect outfall south of Key West High School that flows into the Salt Ponds. As-built drawings were used to include this new pump station in the 2021 model. This project varies from the pressurized wells because there are no wells and it directly pumps to the outfall.

2.3.4.7 Hydrologic Characteristics

The SCS methods referred to in Section 2.3.1 included using the Curve Number Method to estimate excess runoff volume and a unit hydrograph to predict the timing of runoff. The peaking factor used for the unit hydrograph was 256, which is commonly used by the SFWMD for near-flat landscapes. The City is mostly built out and there have been no major land use changes on the main island since 2005, so the same curve numbers from the 2005 City model were used. Table 2-6 lists those curve numbers used previously. By land use, these curve numbers are fairly high (more runoff volume), but the City has a fairly high impervious area because of the relatively high density on the main island. In addition, the runoff potential is greater because of high groundwater levels relative to the ground surface. The time of concentrations for the unit hydrographs were adopted from the City's model that was based on the SCS TR-55 method.

Table 2-6. Curve Numbers Applied in the City of Key West

Land Use	Percent Impervious	Curve Number
Residential High Density	60	91
Open Land	0	80
Retail Sales and Services	84	95
Residential Medium Density	45	88
Commercial and Services	79	94
Recreational	14	83
Institutional	73	93
Industrial	80	94
Mobile Home Units	65	92

The hydraulic elements refer to those physical facilities that are designed to move stormwater and also include the overland flow that may occur when the pipes are too small to convey the runoff flow rates. Stormwater runoff reaches a node and then can stage up if the capacity of the inlets (also called the catch basins) are insufficient to allow the water to enter the pipes. If the pipe capacity is overwhelmed, then runoff also can stage up over the inlets in the streets until water starts to flow to lower elevations, very often through the streets. As noted in the beginning of this section, inlets are normally sized to exceed the capacity of connecting pipes so it is assumed in the H&H model that all stormwater can get into the pipes. This may not be the case if the inlets are blocked with debris or, as is likely in many parts of the City, if the inlets are relatively small. It is assumed in the City's model, as is typically done, that inlets are not limiting flow into the system, and are simulated in a maintained, free-flowing condition.

In coastal regions, there are two methods commonly applied to analyze stormwater systems: by sub-basin or by interconnected sub-basins. The sub-basin approach allows water to stage up only within the sub-basin until the pipes or other infrastructure can drain the stormwater. This approach is applicable when sizing elements to manage runoff from a limited area. The disadvantage of the sub-basin approach for a regional plan is that different sub-basins will stage up to different heights, and flood mapping will be discontinuous. For Key West's 2021 Stormwater Master Plan, the sub-basins were modeled interconnected by the streets or other low areas, which also was the approach used in past studies. The street connections were simulated in the 2012 model as a typical two-lane street channel of irregular shape. In some instances, the streets were modeled as overland weirs with similar roadway shape.

As part of the scope, two-dimensional (2-D) modeling was considered. The ICPR4 model has a new feature that allows for a 2-D computational method. After review of the new DEM data and considering the state of development already present in the City, it was determined that there was no advantage in using the 2-D model. Additionally, the 2-D module of ICPR4 generally is suited for broad expanses of open, low-lying- areas to better represent the spread of water in overland flow systems. Interconnecting the sub-basin with street channels better represents most runoff flow in the City. Although the LiDAR data were originally reduced to be accurate for a 3-meter resolution, the NOAA DEM was resampled at a 5-meter resolution (16.4 feet), which would not capture the streets or other objects any better than was accomplished in the past work. The time to simulate storms and the file sizes become very large (almost unwieldly) when the 2-D option is used. Consequently, only one-dimensional modeling was conducted.

Storage to hold the excess runoff while it is being routed through the pipes or streets was determined by using the DEM. Elevation area was exported from the GIS and entered into ICPR for the nodes.

Pipes included in the model mostly represented the main conveyances toward each outfall or recharge well. The invert elevations typically were the same used in the existing conditions model adjusted to NAVD 88 unless there were new as-built data for the new construction. Pipe diameters in the 2005 model were checked against the GPS inventory in 2012.

The boundary elevation at the outfalls used in the Master Plan was the same elevation reported for the Key West tidal gauge for the MHHW, a constant 0 NAVD 88. Again, studies can assume different boundary conditions, either constant or varying tides. In varying the tides, an assumed sinusoidal curve is used to represent boundary elevations. To be conservative, the timing of the curve would be such that the peak would cause the highest flooding on land. Alternatively, by assuming a constant high tide elevation, the timing of the tides is not an issue. However, the constant boundary condition assumption in the model will simulate long-duration flooding, which is overly conservative. Regardless, both approaches should provide similar peak flood elevations.

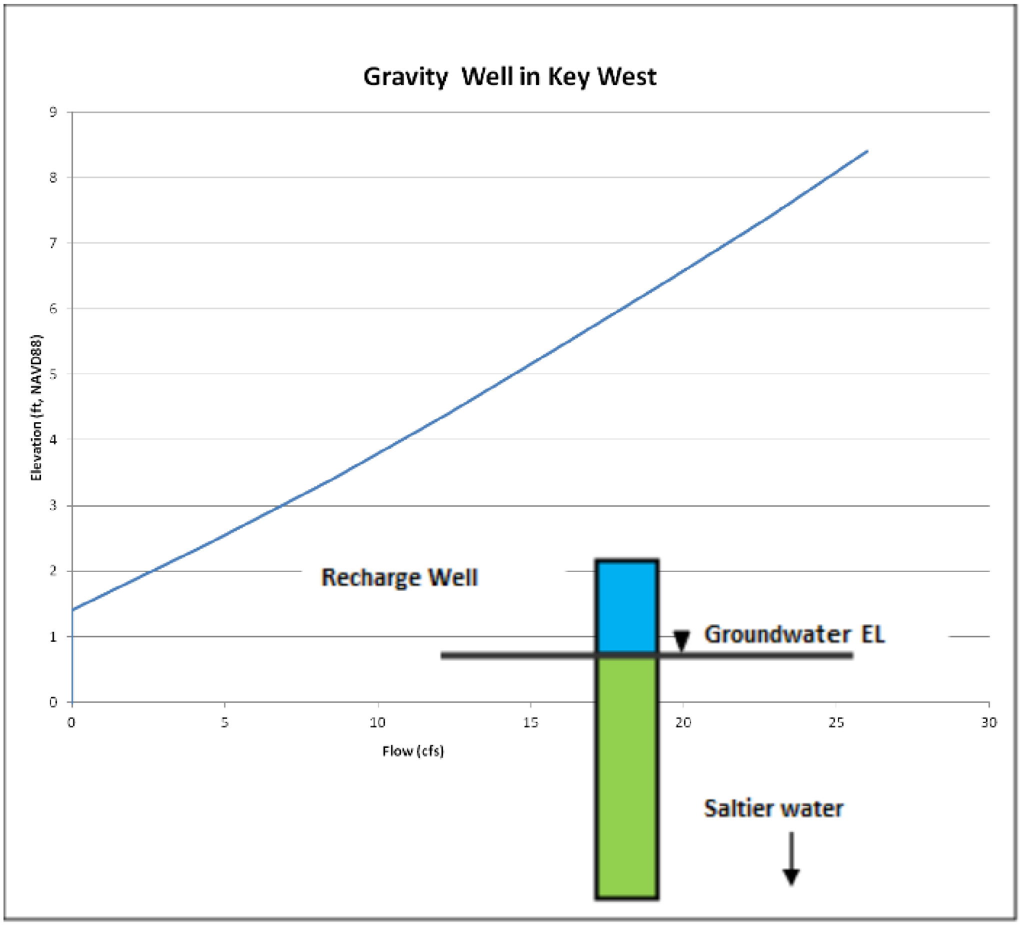
2.3.4.8 Recharge Wells

There are two types of recharge wells used in Key West to manage stormwater, those flowing by gravity or by pump assistance. In general, when the landscape is less than approximately elevation 2.7 NAVD 88, the capacity of the gravity-driven wells is diminished by the high groundwater conditions and limited depth of staged stormwater over the top of the well. Stormwater (freshwater) must build up to overcome the density difference of the saltier groundwater so flow down a well cannot begin until water above the casing reaches 1.4 feet deep. Friction losses will require another 0.2 foot of water, so flow does not begin until stormwater stages to approximately 1.6 feet NAVD 88. Because the groundwater table normally fluctuates with the tide, a conservative estimate is to assume that the elevation of the groundwater in the well is approximately high tide (elevation 0 NAVD 88, MHHW) and would stay at high tide during the entire storm. These wells are typically 24 inches in diameter, cased to approximately 60 feet deep, with the open hole extending 90 to 120 feet below land surface. These wells sometimes are referred to as shallow recharge wells, as opposed to the deep injection wells used at the wastewater plant, or more simply as drainage wells. However, City operations staff also sometimes label vertical French drains as shallow wells, but these are often open-bottom inlets with a short section of perforated pipe (or nothing but soil) to reduce ponded water between storms. These vertical French drains are not included in the model because they are not expected to relieve high peak runoff rates.

The flow down the recharge well also will depend on the ability of the rock formation to accept additional flow volumes. In general, the limestone under the City is highly transmissive, so much so that it is sometimes difficult to take physical measurements of the capacity. In 2002, CH2M prepared a white paper about recharge well capacities where data from recent wells were reviewed and the rating curve on Figure 2-9, adjusted for NAVD 88, was recommended. This rating curve has been used to represent all gravity wells in the City since that time. This rating curve was relatively conservative (that is, low flow) compared to the data, but recharge well performance will reduce over time, and the wells require maintenance to remove debris. The City does routinely clean the stormwater wells, but fines can still reduce some porosity. There are also some instances where the limestone was not as porous as typical, but in general the standard rating curve was used uniformly in the master plans.

As SLR changes the groundwater, the curve shown on Figure 2-9 will shift to the left. Existing gravity recharge wells, especially those located on low landscape, will become less effective with time.

Figure 2-9. Gravity Recharge Well Rating Curve Used in the ICPR4 Model

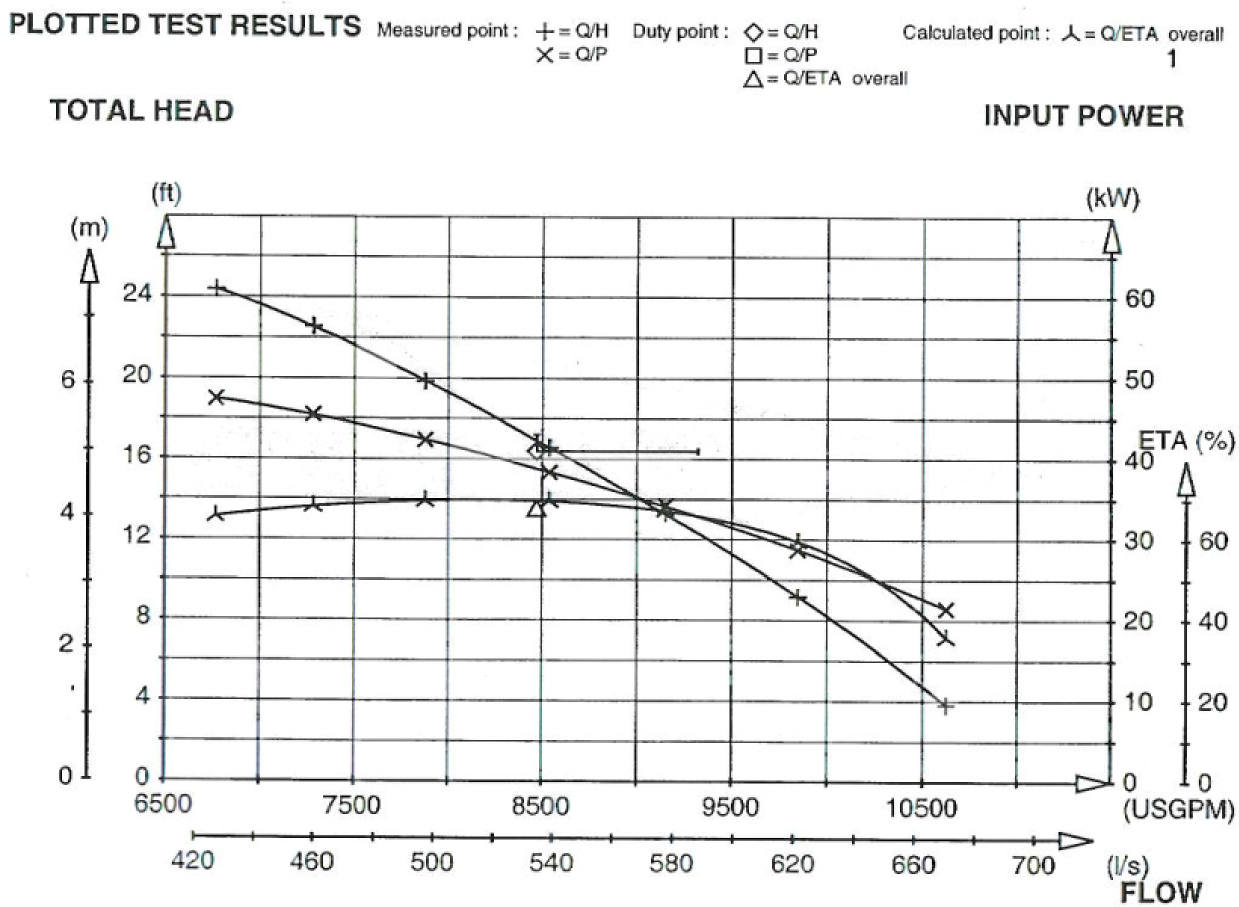


There are four pressure-assisted wells systems in the City: Patricia and Ashby Streets, Simonton Beach, George Street, and White Street. Simonton Beach, George Street, and White Street have two wells, while the Patricia and Ashby Streets facility has one well. Stormwater is treated in vortex separator units and then pumped to the recharge wells, except at George Street, where a water quality box is used prior to the pump station. These sites operate at low landscape elevations, and the pumps provide the extra force to push the freshwater down the well. The pressure wells themselves are the same size and depth as the gravity wells, but the pumps and wells are matched such that the capacity of the pump station matches the well capacity. Each pump/well combination was rated at approximately 8,300 gallons per minute, or 18 cubic feet per second. While the rate will vary somewhat between sites and as the water levels change, the pumping rate was assumed constant for the master plan models.

From Figure 2-9, one can infer that the net pressure at the pressure-assisted wellhead is approximately 6.2 feet, but that can vary slightly between sites depending on many factors (for example, number of pipe elbows and splits in the piping, valving, actual factory impeller rating, and actual groundwater elevations at the well). In fact, the actual operation of the Simonton Beach pressure-assisted system has been at a higher pressure than expected, likely from the poor permeability of the limestone at that location. Because of this, it was assumed that the actual capacity of the existing system was only approximately half (equivalent to one well) for simulating existing conditions. Regardless, a typical pump curve that CH2M used for the design is shown on Figure 2-10. The range of flow varies only by a small rate over a few feet of water level change so a constant discharge rate of 18 cubic feet per second was used for the rating curve.

Figure 2-10. Pump Curve for Flygt Pump Used at Patricia and Ashby Streets Facility

SN: 7050.680 0361043, Impeller No. 428 42 16



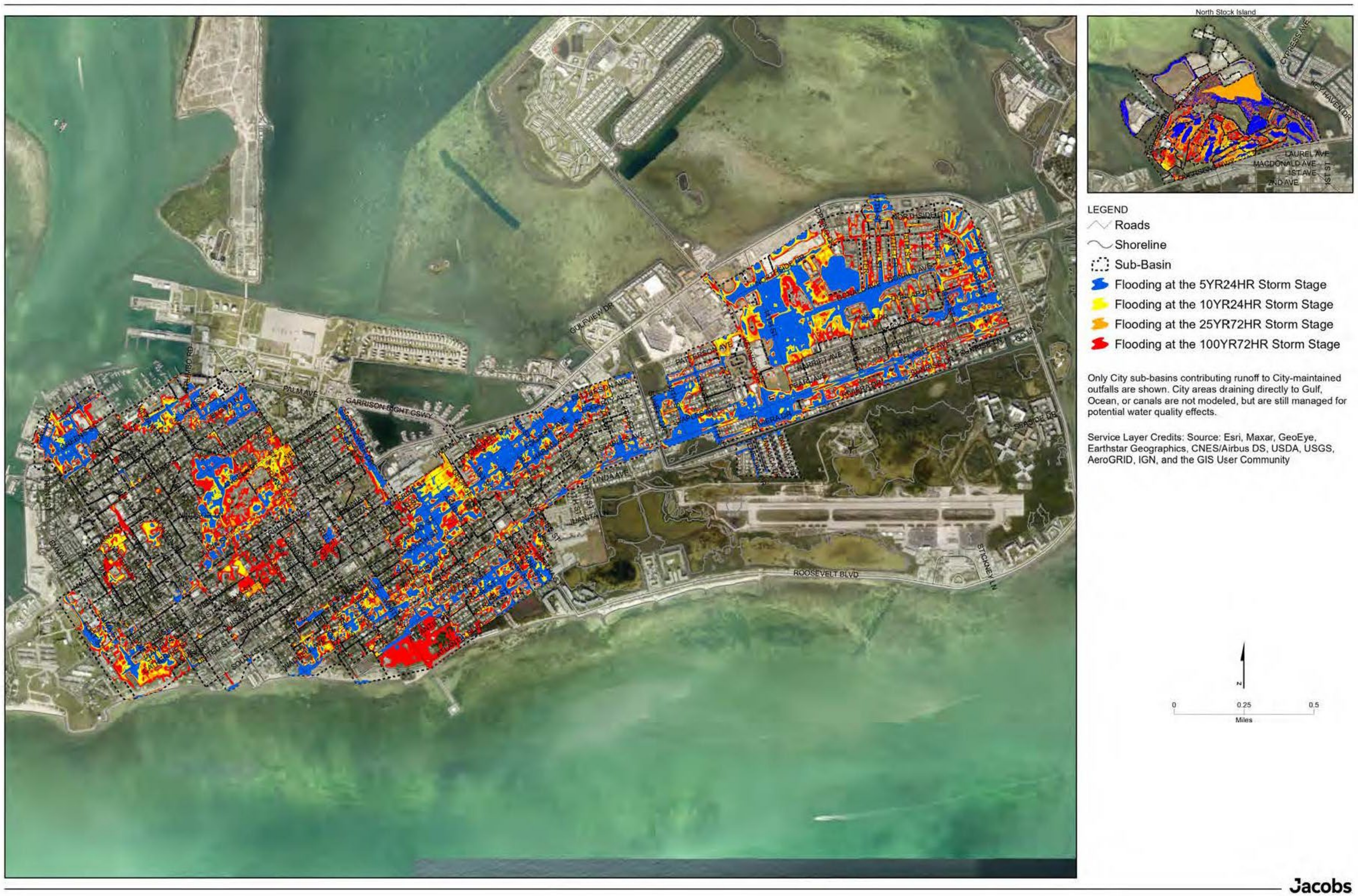
2.3.4.9 Design Storms

The design storm sizes were listed in Section 2. The ICPR4 model has the SFWMD time distributions of rainfall intensities included as standard options. The global storms option was used to simulate the four storms: 5-, 10-, 25-, and 100-year return period storms. By SFWMD convention, storms smaller than 25 years were of 24-hour duration and the two larger storms (25- and 100-year) had a 72-hour duration. Using SFWMD's guidance, the longer-duration storms have constant rainfall for 48 hours, then the intense 24-hour storm occurs. This approach is used in Florida so that stormwater facilities have a factor of safety in their capacity to manage the flood volume from large storms during wet season.

2.4 Existing Conditions Modeling Results

The stormwater model estimates the staging of stormwater at nodes in the model. The elevations of the peak staged levels of stormwater runoff were plotted on the topographic map for the four design storms, as is shown on Figure 2-11. The runoff stages primarily in the lower landscape areas, as has been documented previously. These results were used as a basis for evaluating new projects later in this report. Appendix A provides the predicted peak stages for each node in tabular form.

Figure 2-11. Existing Flood Map



2.5 Preliminary Assessment of Existing Flooding Conditions

This section describes how the information and data provided in the previous sections were applied to develop and evaluate specific projects. The existing flooding conditions were presented in Section 2.4. To develop projects for the City capital improvement plan, the same three-step process that was used in the 2012 Stormwater Master Plan was followed:

1. Identify areas with larger flooding issues (that is, rank the sub-basins where excessive flooding occurs)
2. Evaluate projects for these areas to determine their effectiveness
3. Assess the potential projects to provide a priority for implementation

This section provides a preliminary identification of areas with larger flooding issues. Additional work in identifying projects was conducted later for a limited number of locations (Section 3).

Key West has some unique characteristics that affect the way stormwater projects are prioritized. For example, the low elevations near the coast make traditional "pipe" projects less effective unless very large pipes are used. The highly developed island does not normally have sufficient area for large pipes, especially considering the other utilities. The City wants to reduce stormwater pollutant discharge into the nearshore coastal waters to help protect the natural resources, including beaches, and larger conveyances alone will not achieve this water quality goal. Residents are accustomed to standing water immediately after a larger rainfall and are normally tolerant if the runoff percolates or drains relatively quickly. Consequently, improvements to drainage are measured in sub-foot improvements and the ability to recover after the peak of the storm passes. However, as SLR increases and persistent property flooding occurs, the City is reconsidering how to deal with the chronic flooded areas.

A methodological ranking procedure that considers flooding issues equally regardless of the area is preferred. When defining and prioritizing projects, applying a strict benefit-cost comparison often tends to skew projects toward high-value neighborhoods that may cause some social justice concerns. However, federal FEMA funding often requires a positive benefit-cost ratio to justify grant funding. Some master plans use a ranking procedure to include water quality values. This procedure tends to highlight highly developed sub-basins as higher pollutant sources because of high runoff volumes; however, because the entire island is mostly built out in moderate to high density, this criterion is not necessary. The City's preferred technologies to reduce flooding include the recharge wells and infiltration best management practices (BMPs), so water quality benefits will be included in the projects. New or larger outfalls will be considered only when other options to reduce flooding are limited.

2.5.1 Identification of Areas with Significant Flooding

Figure 2-11 and Appendix A provided the estimated flood levels for the existing stormwater system. These results were used to identify which sub-basins have more problems than others under existing conditions. This initial assessment and sub-basin ranking is useful for discussion purposes but does not in itself identify priority projects. The selection of priority projects will be conducted with further input from the City.

"Level of Service" (LOS) is a common term often used in drainage studies to evaluate performance. For example, the SFWMD requires that local roads and parking lots drain at least a 5-year design storm, so a typical local road design would provide at least a 5-year LOS for flooding. Some stormwater plans use a scoring system to rank areas. Typical scoring criteria may address the following issues:

- Emergency structures operational during a 100-year flood
- Number of buildings or parcels with high water levels
- Structures (residential and commercial) should be damage free during the 100-year flood
- Length of major roads under target depths of standing water
- Major evacuation routes should be passable during the 100-year flood
- Major streets should be passable in the 10-year flood
- Residential streets should be passable during the 5-year storm
- Length of canals or ditches flooded out of bank
- Pounds per year of pollutant loads

The ranking in the 2012 Stormwater Master Plan was based on CH2M's experience in defining and conducting these types of assessments in coastal areas. Two criteria tend to differentiate projects most often: number of buildings or structures flooded during the 100-year flood and length of major streets flooded in the 10-year flood. Consequently, only these two criteria were carried over to this update.

To determine what constitutes damage to structures during the 100-year flood, the first-floor elevations of each structure would need to be known and that information is not often generally available. Some buildings would be damaged in lower floods, while other buildings are elevated. This assessment assumed that flooding more than 1 foot deep over the general landscape elevation (from LiDAR data) would potentially damage a structure. The Monroe County property assessor parcel database was used to identify lots. To be slightly more conservative, the assessment counted the parcels where there was some 100-year flooding greater than 1 foot deep, even if only a small part of the lot was that deep. Figure 2-12 shows the 100-year flood and the zones deeper than 1 foot (darker red). Note that North Stock Island residences (in the golf course development) were permitted with first-floor elevations set more than 1 foot above the ground so North Stock Island was not included in the sub-basin ranking (no known problems).

The second criterion, length of streets not passable during the 10-year flood, was assessed to all roads in the sub-basins regardless of whether they are considered major. The Monroe County street GIS database was used to identify roads. While some studies may apply a higher value, a 6-inch threshold was selected as a reasonable depth that may last for a short time during a large rain event given the wide variety of roads on the island. The 6-inch centerline depth also is commonly used to test emergency vehicle access. The LiDAR data were used to determine centerlines deeper than 6 inches (0.5 foot) during the 10-year flood. The length of road was determined using GIS to intersect the road centerlines with the flood polygons. Figure 2-13 shows the roads where flood staging occurs at depths greater than 6 inches. The darker shading shows the deeper staged stormwater.

Figure 2-12. Parcels with 100-year Flooding Greater than 1 Foot Deep

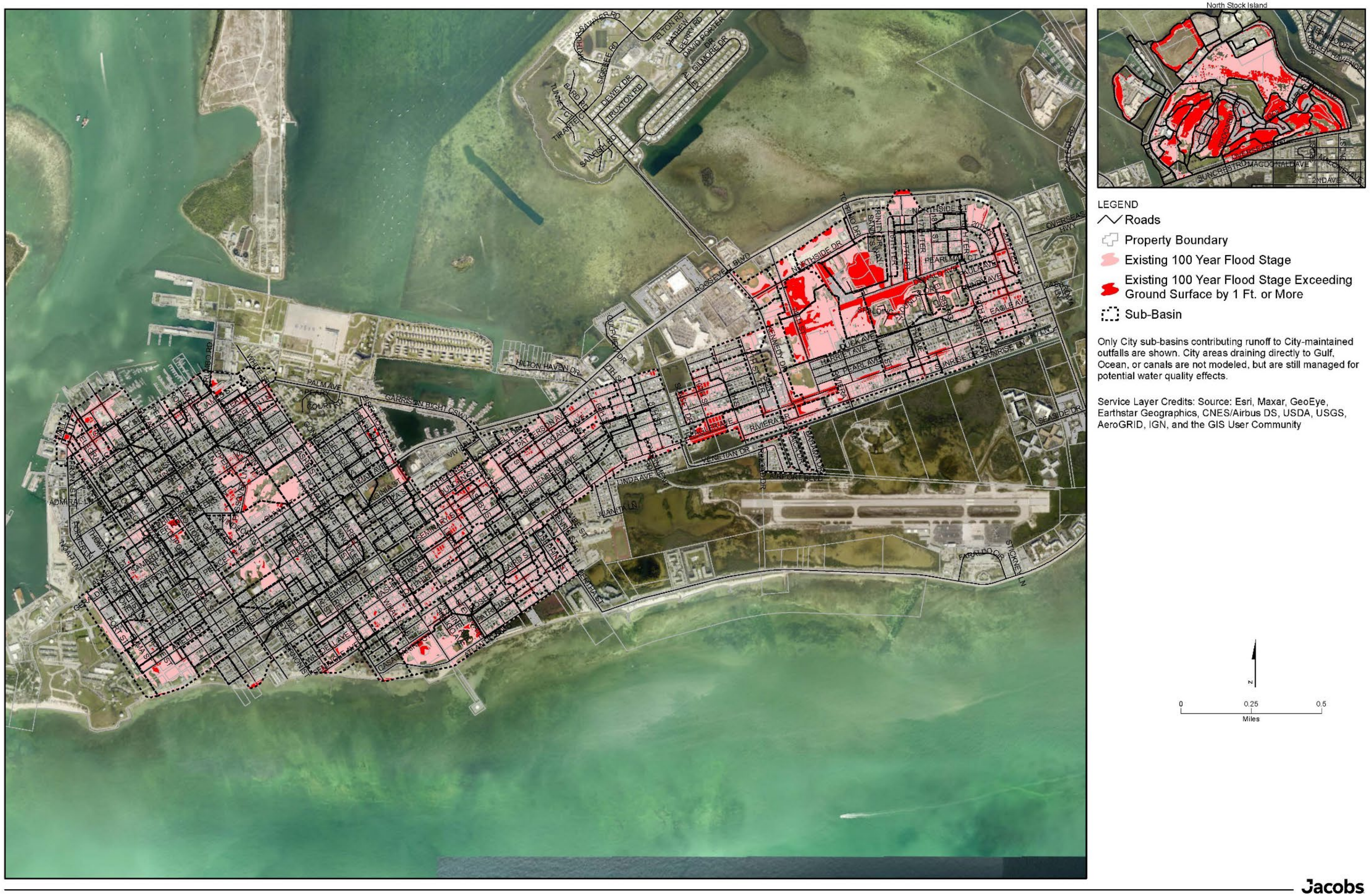


Figure 2-13. Roads with Flood Staging Greater than 6 Inches Deep

Jacobs

To rank the severity of flooding in sub-basins given the two quantitative measures, a simple process was used. The number of parcels per sub-basin with 100-year flooding 1-foot deep was sorted from most to least and then assigned a criterion sub-rank 1 through 137 (out of 229 sub-basins on the main island). The ranking stopped at 137 because only 136 sub-basins had impacted parcels and all parcels with 0 were assigned the same 137 score. No ties were assigned. The ranking was based on the standard sorting routine in Excel with the number of parcels ranked first, then by length of street flooded (10-year simulation, greater than 6 inches deep). Similarly, the length of street per sub-basin was identified and sorted from most to least, and then the road flooding criterion was assigned a sub-rank 1 through 108. The scores of the two criteria were added and then the sub-basins were sorted by low total score, with number of parcels impacted used as a tiebreaker, up to a rank of 156. The remaining sub-basins had the same score. Table 2-7 provides the 50 sub-basins that had the worst flood severity rankings.

Table 2-7. Top 50 Sub-basin Rankings to Identify Areas with Greater Flooding

Sub-basin	Road N-S Reference	Road E-W Reference	Road Length (ft)where 10-Yr Flood Stage Exceeds Ground Surface by 0.5 ft or More	Rank of Length of Flooding in Streets at Least 0.5 ft During 10-yr Storm	Number of Parcels impacted with at Least 1-ft Flooding During 100-yr Storm	Rank of No. of Parcels with at Least 1-ft Flooding During 100-yr Storm	Sum of Scores	Final Sub-basin Ranking
B6000	10th Street	Harris Avenue	1516.97	2	49	4	6	1
B3930	20th Street	Duck to Eagle Avenue	1913.76	1	36	7	8	2
B4147	18th Ter	Donald Avenue	1468.94	3	36	8	11	3
B2830	Thompson Street	Seminary Street	955.41	9	55	3	12	4
B2840	Leon Street	South Street	1037.71	7	37	6	13	5
B2550	Southard Street	Margaret Street	862.85	10	42	5	15	6
B3790	14th Street	Nr. Stadium Apts.	750.96	11	30	10	21	7
B2120	Duval Street	Between Greene and Front Streets	1048.43	6	23	17	23	8
B3610	Between 10th and 11th Streets	Flagler Avenue	1176.99	4	21	21	25	9
B3220	4th Street	Fogarty Avenue	996.07	8	22	18	26	10
B1015	6th Street	Patterson Avenue	665.39	15	26	13	28	11
B3030	Ashby Street	United Street	636.82	17	28	11	28	12
B2510	White Street	Eaton Street	625.89	18	27	12	30	13
B3837	13th Street	Riviera Drive	1059.98	5	18	27	32	14
B4110	17th Street	Donald area	730.64	12	20	24	36	15
B3020	Ashby Street	Catherine Street	511.94	24	26	14	38	16
B2560	Margaret Street	Angela Street	416.43	38	59	2	40	17
B130010	Ashby Street	Rose Street	546.52	23	19	25	48	18
B3600	11th Street	Flagler Avenue to Riviera Drive	584.07	22	18	26	48	19
B4150	20th Street	Cindy Avenue	501.22	25	18	28	53	20
B3800	Rivera Street (15th)	Flagler Avenue to Riviera Drive	447.85	34	20	23	57	21
B2705	Jose Marti Drive/Eisenhower Drive	Truman Avenue	596.65	20	13	39	59	22
B2820	Thompson Street	Catherine Street	481.58	27	16	32	59	23
B3260	2nd Street	Fogarty Avenue	593.55	21	13	41	62	24
B210	White Street	Laird Street	464.11	32	16	31	63	25
B2110	Simonton Street	PS	324.92	45	21	20	65	26

Sub-basin	Road N-S Reference	Road E-W Reference	Road Length (ft)where 10-Yr Flood Stage Exceeds Ground Surface by 0.5 ft or More	Rank of Length of Flooding in Streets at Least 0.5 ft During 10-yr Storm	Number of Parcels impacted with at Least 1-ft Flooding During 100-yr Storm	Rank of No. of Parcels with at Least 1-ft Flooding During 100-yr Storm	Sum of Scores	Final Sub-basin Ranking
B3200	4th Street	Patterson Avenue	700.75	13	10	53	66	27
B605	Emma Street	Amelia Street	456.59	33	13	45	78	28
B3760	13th Street	Northside Drive	465.96	31	11	50	81	29
B2520	Frances Street	Eaton Street	430.76	36	11	48	84	30
B3830	Between 13th and 14th Streets	Flagler Avenue	656.05	16	8	68	84	31
B3115	1st Street	Patterson Avenue	603.02	19	8	66	85	32
B3400	8th Street	Flagler Avenue	469.01	30	9	56	86	33
B2120b	Ann Street	Greene Street	469.25	29	8	63	92	34
B4120	18th Street	Donald Avenue	324.65	46	12	46	92	35
B240	Whalton Street	Von Phister Street	169.14	59	14	35	94	36
B3620	12th Street	Flagler Avenue	385.95	39	9	58	97	37
B3210	3rd Street	Patterson Avenue	232.91	51	11	49	100	38
B2400	Margaret Street	Caroline Street	429.42	37	8	64	101	39
B2100c	Fitzpatrick Street	Front Street	678.54	14	4	88	102	40
B3820	14th Street	Flagler Avenue	334.81	43	9	59	102	41
B130	Ashby Street	Patricia Street	495.42	26	5	78	104	42
B3912	18th Street	Riviera Drive	445.47	35	7	70	105	43
B4145	19th Street	Cindy Avenue	469.95	28	6	77	105	44
B130012	Ashby Street	Johnson Street	210.40	54	9	55	109	45
B200	White Street	Between Atlantic Blvd. and Casa Marina Court	5.77	101	32	9	110	46
B3810	16th Street	Flagler Avenue	71.59	74	14	36	110	47
B130005	Josephine Street	Atlantic Blvd.	161.52	60	9	54	114	48
B4130	20th Street	Donald Avenue	60.49	77	14	37	114	49
B400	Alberta Street	Seminole Avenue	359.79	40	6	76	116	50

Previously, some sub-basins ranked higher because of their relative size. For example, Sub-basin 130 scored somewhat higher than others in 2012 because the entire Kamien neighborhood has extensive peak stormwater flooding resulting from the low elevations (many affected parcels). Because of the design of improvements during the past 8 years, the modeling was further refined to include more detail (smaller sub-basins), so the smaller new sub-basins in this neighborhood moved down in the 2021 ranking because of the number of parcels and length of roads impacted with split from one to 11 sub-basins.

2.5.2 Identification of Potential Projects

The ranking of sub-basins is a tool that the City can use to identify areas that are likely to have more severe flood issues. However, the rankings need to be evaluated based on additional information, including consideration of past projects and areas of potential new development. The low areas around the coast continue to be an issue because of the low elevations and increasing vulnerability to rising ocean levels.

3 Project Identification and Evaluations

Phase 1 funding for the current SWMP update included the work described through Section 2. Phase 2 funding used the new ICPR4 model results to select areas where future stormwater projects could move into design. Inundation levels were reviewed, and LOS metrics related to length of roads flooded during a 10-year event and parcels experiencing flooding during a 100-year event were identified to develop a preliminary ranking as described in the previous section.

After the 2021 evaluations were completed, the City conducted a review of climate change considerations and has considered developing specific, design-related policy toward SLR. An SLR policy report can provide a framework and guide that incorporates SLR projections into infrastructure design criteria. These new policies and design criteria have not been adopted by the City Commission, but they create a baseline to build upon for future City capital investment and for possible use in future development requirements.

The Jacobs 2021 SLR report (Jacobs 2021b) provided recommended minimum design criteria. In addition, the recommendations for stormwater boundary conditions were anticipated to be used to inform the ongoing 2024 SWMP update and BMP analysis. The higher boundary conditions were applied to develop new recommendations and formulate how future projects may need to change to cope with higher ocean levels. For planning purposes, the City suggested using the 1-foot, fall MHHW plus the Southeast Florida Regional Climate Change Compact projections (Compact 2020). This was discussed in Section 2.2.2 of this report.

The updated 2024 SWMP reviewed all sub-basins within the City, and areas with the worst flood inundation were similar to the 2012 evaluation. The City needs are extensive and funding for capital projects is limited. Consequently, Jacobs' scope of work was to develop conceptual designs for seven high priority- sub-basins. After the 2021 simulations were completed, select priority areas were identified for more-detailed evaluation during this update. For example, some areas already had designs under way, so the SWMP did not review additional projects for these areas. Some of the low areas encompassed more than a single sub-basin, so the City agreed to look at five neighborhoods or low areas in greater detail, which exceeded seven sub-basins. In 2023, the City asked Jacobs to include a potential stormwater project in the southern Bahama Village neighborhood. Specifically, this 2024 Update was to identify the types of projects needed to address potential SLR in the next 30 years in these selected study areas. The results of this SWMP help the City to recalibrate its expectations for the type of projects needed to maintain resiliency to climate change.

North Stock Island is part of the City, and modeling was included in the 2021 SWMP update. However, no priority projects were identified on Stock Island. The 2012 SWMP noted that the outfalls servicing College Road need regular maintenance and debris cleanout to prevent clogging by the mangroves. The City installed inlet rack inserts to capture debris and sediment and serviced the outfalls with mangrove trimming. These operations and maintenance (O&M) services must continue, even though additional projects have not been identified for North Stock Island at this time.

3.1 Conceptual Approach to Defining Projects

The study areas selected for the Phase 2 conceptual solutions included locations from the previously prioritized sub-basins identified in Phase 1. The priority rankings conducted during Phase 1 were by sub-basin. Larger areas where more than one adjacent sub-basin was considered higher priority were grouped into "zones." These zones were developed for report organizational purposes and are not intended to represent any current City planning or neighborhood naming convention. Figure 3-1 illustrates the zones and study areas (that is, priority sub-basins) evaluated as part of this phase.

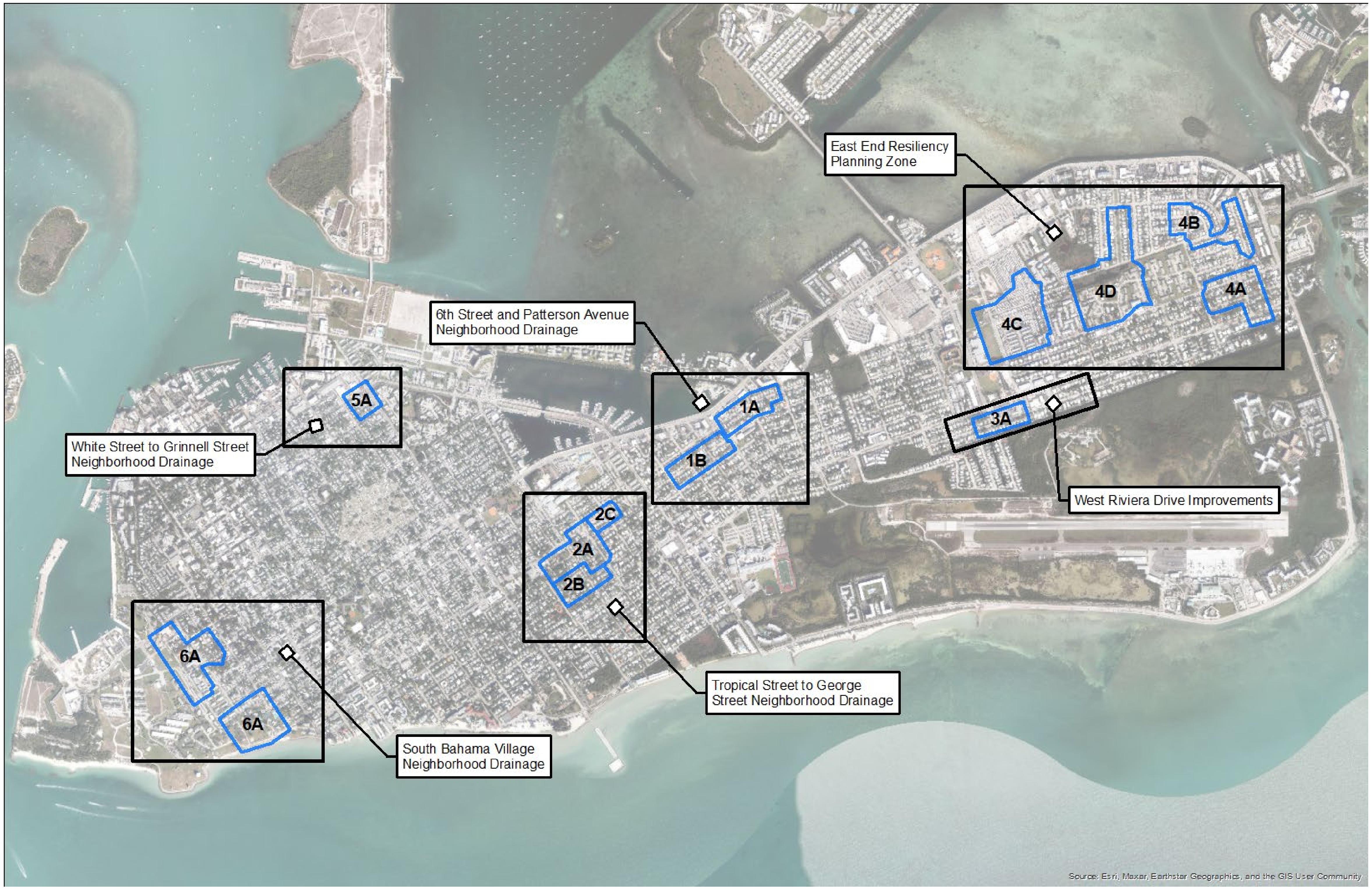
Each zone encompasses one or more sub-basins. The numbering of each study area is a relative priority rankings. Not every top-ranked sub-basin was included in the list because they were not selected for further study. Conversely, southern Bahama Village was added later and was listed last. The zones were based on the larger groupings where sub-basins interacted with each other, so more than seven sub-basins were included in the project evaluation.

3.1.1 Study Areas

Study areas are described as follows:

1. 6th Street and Patterson Avenue Neighborhood Drainage
   a. 2nd Street to 5th Street between Patterson Avenue and Harris Avenue
   b. Patterson Avenue between 5th Street and 7th Street
2. Tropical Street to George Street Neighborhood Drainage
   a. Tropical Street to Thompson Street between Duncan Street and Washington Street
   b. Washington Street between Tropical Street and Thompson Street
   c. Duncan Street to Seminary Street between Thompson Street and George Street
3. West Riviera Drive Improvements
   a. Riviera Drive Improvements
4. East End Resiliency Planning Zone
   a. Eagle Avenue at 20th Street
   b. 18th Terrace at Donald Avenue east to 20th Terrace
   c. Glynn Archer Jr. Drive between Glynn Archer Jr. Street and Duck Avenue
   d. Northside Drive to Duck Avenue between 15th Terrace and 17th Terrace
5. White Street to Grinnell Street Neighborhood Drainage
   a. Frances Street to White Street between Eaton Street and Fleming Street
6. Southern Bahama Village
   a. Specific request to address flooding near two community centers at Olivia Street and Emma Street (gym) and Catherine Street and Thomas Street (park with pool).
   b. The contributing sub-basins are bounded to the east near Whitehead Street by the U.S. Navy facilities west of Fort Street, and Angela Street to the north.

Figure 3-1. Priority Sub-basins



The existing conditions throughout the targeted study areas include low-lying road intersections and public-use areas; limited drainage infrastructure and outfall capacity (small pipes); limited slope (poorly drained landscapes); and property constraints that limit the effectiveness of storm sewer improvements in the target areas. Traditionally, the City improved drainage by installing gravity-driven drainage wells (that is, Class V underground injection control permitted wells) that push freshwater approximately 60 to 120 feet below ground when staged stormwater runoff filled intersections. Many intersections in the City have one of these types of "gravity wells" tied to several inlets around each corner. Some areas are too low for gravity wells to function at high capacity. During the 2012 SWMP, the effectiveness of gravity-fed injection wells was reviewed, and it was recommended to avoid adding these structures at elevations lower than 3 feet NAVD 88. SLR and high-tide scenarios, such as those that contribute to sunny-day flooding, will increasingly limit the effectiveness of stormwater outfalls and gravity injection wells that currently exist at many locations. These rising boundary conditions also limit the effectiveness of stormwater infiltration and storage opportunities throughout the City.

At four locations, the City's pressurized injection well systems can serve larger contributing drainage areas and work at low elevations. Each system has an emergency bypass outfall to the ocean. These pumped systems are effective in high ocean and groundwater conditions. A treatment system is required prior to the pump station, like a vortex unit or advanced settling basin. Resilient solutions for future conditions must focus on conveyance and outfall improvements, coupled with stormwater pump-based options.

3.1.2 Changing Boundary Conditions

It is prudent for the City to plan for some level of future SLR for resiliency purposes. The City evaluated the new tide data and recommendations from others and developed a draft SLR policy that sets certain target elevations for future infrastructure (Jacobs 2021b). In general, the policy considered the service life and criticality and may include a factor of safety. In practice, designing new infrastructure to blend into the existing residences and buildings is important and it may preclude raising roads or adding sea walls as much as desired. This SLR policy is available as guidance. Actual conditions and existing infrastructure may alter the design of specific projects.

Drainage and roads typically have a service life of approximately 25 to 35 years. This means 2050 is a typical planning horizon and the mean sea level at that time may be approximately elevation 1.15 feet NAVD 88 per the Compact intermediate high projections. The City Policy also recommends using the fall mean higher-high tide of approximately elevation 1.0 foot NAVD 88 for design of existing drainage facilities, so these two elevations are similar. The policy also recommended a sea wall barrier to be as high as elevation 5.0 feet NAVD 88, based on a 50-year service life and a 1-foot factor of safety.

SLR also will affect the groundwater levels in the City, given how the coral rock base of the islands allows direct interaction with the ocean. While the groundwater moves up and down with the tide at most locations, the rise and fall is muted and is expected to average higher than the mean water (ocean) level. For this update, the future groundwater levels were assumed to be 2 feet NAVD 88 across the island. This parameter will change the drainage performance of the groundwater wells (reduce capacity).

The SLR was applied to the MHHW elevation to obtain the proposed future stormwater tidal boundary condition elevation. Because the City is looking at the average fall season MHHW, a boundary condition of 1 foot NAVD 88 was simulated to assess near-term conditions for alternative comparison purposes. With 1.7 feet of SLR, the long-term ocean boundary condition was set at an elevation of 2.7 feet NAVD 88, with a groundwater elevation of 2.0 feet NAVD 88.

The selection of study areas for further evaluation was based on the flooding encountered under existing mean tidal boundary conditions, and these same boundary conditions were used to test the size of and refine conveyance improvements. When preferred conveyance routing and sizing were identified, the solutions were tested with both near-term (1-foot NAVD 88) and long-term (2050, 2.7-foot NAVD 88) tide scenarios to further develop the proposed improvements so they continue to provide benefits under future SLR. This technique helps avoid proposing oversized pipes to resolve flooding in a specific study area unless they also may provide a suitable LOS at the 2.7-foot NAVD 88 tidal boundary conditions.

As the tidal boundary condition is set to future conditions with the improvements in place, the improved study areas may receive more overland flow from the surrounding flooded sub-basins, limiting the benefits. Therefore, a regionalized approach that would contain resiliency-based concepts is needed to compartmentalize improvement zones, so improved areas function to the greatest extent possible (such as regional pump stations, potential road raising along many sections of roads citywide, and other innovative and non-traditional stormwater management measures).

Several conceptually modeled solutions were examined to develop a preferred alternative for each study area. Preferred solutions are those that were identified as the most cost-effective while providing an increased LOS during the 5-year or 10-year, 24-hour rain events. Efforts were made to develop solutions for greater than the 10-year rainfall events, but the general target for service along City roadways was either the 5-year or 10-year event under 1-foot tidal conditions, as well as the regionalized, resiliency based- solution approaches for the 2.7-foot tide boundary condition.

3.1.3 Study Area BMP Solutions Analysis

There are two challenges to provide substantial drainage relief in the lower topography elevation zones in Key West:

- Conveyance to an outfall
- Outfall capacity with high boundary conditions

The conveyance between low spots is primarily through streets or the limited pipes (limited by size, slope, and extent). Often, stormwater stands in the curb areas until percolation occurs. In some areas, the rights of way are mostly paved, so percolation is restricted. There are also shallow well vaults (vertical French drains) and small inlets with open bottoms to facilitate percolation in a localized fashion. The outfall capacity often is either to the ocean outfalls or to gravity-driven wells. However, as noted in 2012 and herein, the groundwater will rise with SLR, and that will limit the flow rates down the gravity wells. Higher ocean levels also will limit the amount of flow through the outfall pipes, regardless of size. When considering future resilient solutions, much more extensive projects will be required.

With ocean levels approximately 1 foot to 2.7 feet NAVD 88 and higher groundwater, gravity-driven flow will not work at many locations. More pump stations will be required. These pump stations may be directed to pressurized wells and outfalls. Relying on the streets or an undersized pipe system also will become less effective with higher groundwater, so larger pipe networks are required to move the runoff to the pump stations. The proposed projects include both of these elements. It may be possible to phase the implementation of the projects in neighborhoods. For example, better conveyance to a larger outfall would help now. As SLR occurs, a pump station will be required in the future. Some zones would benefit from pump stations sooner than others.

When analyzing the conceptual solutions with future tide boundary conditions, several assumptions are required to size the facilities. To plan for these conditions, the following modeling conditions were used:

- The anticipated design of these solutions must consider given SLR conditions of 1.0 foot NAVD 88 and 2.7 feet NAVD 88 for near term and long term, respectively. Some of the existing stormwater features would be completely inundated by the tidal conditions alone. All stormwater outfalls are assumed to have flap gates or check valves installed that will prevent high-tide conditions from flowing directly into the low-lying landscapes through the stormwater conveyance system.
- The proposed conveyance improvements and stormwater pump stations are conceptually sized to remove flooding conditions when subjected to the 10-year, 24-hour storm event with a tidal boundary condition of 2.7 feet NAVD 88. This LOS was selected because much of the island that is currently lower than that elevation will no longer be able to rely on gravity flow, infiltration, and other current means to dissipate flooding conditions. As such, flood duration and depth will continue to increase, creating a reliance on these stormwater solutions and the potential need to modify and upgrade them over time to meet continually changing conditions.
- All stormwater gravity injection wells are assumed to operate at a reduced capacity based on changes to the rating curve that considers rising groundwater conditions. The modeled flow condition in these wells is set to positive only, assuming that ineffective wells will be plugged over time. As groundwater rises, these wells may begin to contribute to standing water in the intersections, so they will be obvious. For example, most of the gravity wells in the Kamien neighborhood have been abandoned so groundwater would not flow into the new storm sewers. This is possible because the new drainage pipe network leads to a pressurized injection well system. For simulation purposes, gravity wells that become ineffective are assumed to not work, or work only marginally, after water stages deeply.
- Aside from the stormwater outfalls, there may be areas of the island, such as along Riviera Canal south of Riviera Drive, that are below the future tide boundary conditions. It is assumed these areas will be managed under the draft SLR guidance document (Jacobs 2021b), and either seawalls or roads that discharge through overland flow to these areas will be raised to prevent sunny-day flooding, accordingly. The cost opinions included in this report do not include costs for these regional sea barrier measures.
- The proposed solutions assume that raising the roads in residential areas will not be practical at this time without a high degree of planning and preparation. Target flood relief for these areas assumes only up to 6 inches maximum of road raising may be applicable without detailed survey information available to confirm. A raised-road condition was not modeled explicitly. The study areas have some of the lowest landscape elevations, and though raising the road may decrease some storage within the sub-basin, it would also reset the LOS within them to the new top-of-road elevation. A higher road also may marginally improve conveyance through the stormwater network servicing the area. In other words, significant road raising would have to be evaluated on a case-by-case basis during detailed design. Target LOS criteria for this analysis assumes that up to 6 inches of flooding above the low area of the roadway may be mitigated by road raising or just tolerated.
- The proposed solutions assume that, given the projected future tidal boundary condition of 2.7 feet NAVD 88 and the landscape elevations encountered in the study areas, it will be permittable to construct new upsized regionally located coastal outfalls to handle peak-flow conditions for the targeted LOS. Rights-of-way and access issues for these outfalls have not been evaluated for the master plan. Section 4 of this report includes potential regional/resiliency-based concepts, such as a modified, pump-assisted gravity well, that may assist in providing more outfall capacity to the area where space is suitable. These proposed modified gravity wells could be adapted to work with the proposed pump stations included within these study areas for handling lower-intensity, more frequent storm events.

3.1.4 Class 4 Cost Opinion Assumptions for Study Areas

Cost opinions for the preferred solutions have been developed based on historical construction cost information from past stormwater projects in the City, as well as relevant projects throughout the region. Cost escalation factors have been used to project historical project cost data to the current conditions based on Engineer News Record (ENR) cost indices. Likewise, where regional construction data were used from resources such as the FDOT construction cost tracking data, a location factor was applied to adjust for the local market on the island. These values, as well as contingencies and various soft costs, are included in the cost opinions as documented in Appendix C. The scope of work calls for Class 5 estimates, which is appropriate for entirely conceptual level estimates. However, given the amount of thought and study, available cost data for similar items in the Keys, and Florida experience, Jacobs would classify the cost opinions presented as closer to Class 4 estimates with approximately 1 percent design completed. Other cost-opinion-related assumptions include:

- While the modeled solutions focus on conveyance pipe sizes, the Class 4 cost opinions include design related- features such as inlet quantities and auxiliary drainage-related features incidental to the conveyance and outfall elements included in the exhibits. For example, where pipe sizes are increased or new conveyance pipes are proposed, the design features also will include roadside inlets at corners of the intersections to be improved.
- Stormwater pumps are only conceptually identified in the modeled solution simulations. Costs for these facilities are estimated based on recent project history in the City, as well as the south Florida area, and projected based on flow capacity in cfs.
- Resilience-oriented solutions servicing locations outside of the select study areas need to prevent these outside areas from flooding into adjacent study areas after a reduction in flood stage occurs from a project. This issue was considered further in this SWMP under separate, regionally based approaches. Costs for these broader-scale items are included in Section 4of this report.
- Property and easement acquisition are not included in the estimated costs, as the proposed features are predominantly located along local roads and access areas, where the space may be available either within areas already owned and maintained by the City or potentially mutually beneficial to commercial areas.
- Costs included in the estimate may be modified based on the actual implementation schedule, which could increase costs based on factors such as inflation and market volatility beyond that of the 25 percent factor used for this project. These opinions were prepared in early summer 2022, when the ENR Construction Cost Index was approximately 13,110. Because Key West is an island where there are extra delivery and work force costs, mainland data needs to be judged carefully. Using local bid data helps address these factors, but the volatility in bid prices is high.
- The cost opinions are not an offer for construction or project execution. These Association for the Advancement of Cost Engineering (AACE) International Classification Class 4 cost opinions are assumed to represent the actual total installed cost within the range of -30 percent to +50 percent (based on AACE) of the cost indicated. The cost opinion has been prepared for guidance in project evaluation and implementation from the information available at the time of the estimate. The final costs of the project will depend on actual labor and materials costs, competitive market conditions, implementation schedule, and other variable factors. As a result, the final project costs will vary from the estimates presented herein. Because of this, project feasibility and funding needs must be carefully reviewed prior to making specific financial decisions to help ensure proper project evaluation and adequate funding.

Other assumptions and relevant information are included in the individual cost opinions for each study area as presented in Appendix C.

3.1.5 Water Quality Considerations

Local water quality is of great concern for the Keys communities because they rely on these natural resources for many benefits. During the past four decades, the City and other Monroe County communities have strived to improve the water quality in the surrounding waters under the state's stormwater and other preservation programs. The City has an active O&M program to help reduce trash and floatables, which is necessary in such a popular tourist destination. Another strategy to reduce stormwater pollutants was to use injection wells. As these gravity-fed injection wells become less effective, their ability to divert stormwater from local waters will diminish. This change is unavoidable in many locations.

Under the Clean Water Act, Florida assesses its waters regularly and, when there are sufficient data, the waters are considered to have impaired water quality. These impairments are listed by parameter. The Florida Keys were identified previously as impaired for nutrients based on "other information," indicating an imbalance in flora or fauna. However, it was placed in Category 4b (Reasonable Assurance) and was not added to the Section 303(d) list of impaired waters because there was reasonable assurance that it will attain water quality standards resulting from existing or proposed pollutant control measures. The Florida Keys Reasonable Assurance Documentation was approved by the Florida Department of Environmental Protection for nutrients in 2008 and provided to the U.S. Environmental Protection Agency in February 2009. The Atlantic Ocean and Gulf of Mexico surrounding the City are still identified as having elevated total nitrogen. This impairment is being made official and, when done, a total maximum daily load will be required (FDEP 2022). The very nearshore waters around the beaches have concerns about bacteria (beach closings), nutrients, and copper. It is not clear how the impaired waters program will affect the City in the future.

Regardless, each project discussed herein must consider water quality and incorporate BMPs that reduce solids and nutrients to outfalls. Common BMPs include measures such as wet ponds, infiltration methods, and source controls like the inlet racks and water quality boxes. Again, with rising groundwater in the future, infiltration practices such as swales will be less effective in the low landscapes, so more inlet and water quality boxes must be included in the detailed designs.

3.2 Alternatives Evaluation

As noted previously, the general approach for evaluation was to determine effective conveyance solutions with boundary water elevations at 1 foot NAVD 88, with the expectation that pumps could be installed to improve the outfall capacity under higher 2.7-foot boundary conditions. The rationale is that a pump station is expensive, and if it can be deferred, the effort should be focused on improving gravity drainage under current (or near-current) conditions.

The study areas refer to the selected study focus sub-basins presented in Section 3.1.1. The numbering of the study areas does not reflect the ranking during the Phase 1 assessment (Section 2.5).

3.2.1 Study Area 1A, 6th Street and Patterson Avenue

Although Study Area 1A is located just east of Study Area 1B and is in the same Patterson zone, Area 1A has its own outfall to the east, and modeling analysis confirms a benefit from keeping solutions for Area 1A independent of Area 1B.

The proposed conveyance improvements for this area focus on disconnecting the pipe running west down Patterson Avenue to 6th Street and tying it directly to the outfall pipe behind the commercial properties north of Patterson Avenue at 7th Street. Outfall pipes down 6th Street and behind the commercial properties then must be increased in size.

It is important to note that the outfall location to the east presents an additional challenge when compared to the tidal boundary condition of 2.7 feet NAVD 88. The existing roads in the vicinity of the outfall are currently lower than the future tide boundary condition. These areas should be addressed independently as a part of meeting the resiliency standards outlined in the resiliency TM (Jacobs 2021b).

The proposed improvements for the preferred alternative include the following:

- Plug (abandon) existing 15-inch pipe headed west down Patterson Avenue from the intersection of 7th Street.
- Install proposed 190 linear feet (LF) of 36-inch-diameter reinforced concrete pipe (RCP[3]) along 7th Street from the intersection with Patterson Avenue to connect to the outfall heading east.
- Remove 250 LF of existing 12-inch-diameter pipe along 7th Street from Fogarty Avenue to Patterson Avenue and replace with 24-inch-diameter RCP.
- Remove 260 LF of existing 12-inch-diameter pipe along 6th Street from Fogarty Avenue to Patterson Avenue and replace with 30-inch-diameter RCP.
- Remove 180 LF of existing 12-inch-diameter pipe along 6th Street from Patterson Avenue to the outfall headed east along the commercial access area north of Patterson Avenue and replace with 48-inch-diameter RCP.
- Remove 550 LF of existing 12-inch-diameter pipe along the outfall to the east along the commercial access area north of Patterson Avenue to the proposed pump station vault located east of the intersection with 7th Street and replace with 54-inch-diameter RCP.
- Remove 470 LF of existing 24-inch-diameter pipe extending to the outfall location east of 7th Street and replace with 54-inch-diameter RCP.
- Add a proposed 50 cfs (22,440 gallons per minute) peak-flow stormwater pump station located in the vicinity of the commercial access drive north of Patterson Avenue and east of 7th Street; proposed vault to be 440 square feet, extending to elevation -10 feet NAVD 88; pump station to tie into proposed new outfall pipe for discharge piping.

Figure 3-2 identifies the proposed conveyance improvements anticipated through Study Area 1A. The proposed flood stage results are included in Table 3-1.

[3] Alternate pipe material is acceptable. Buoyancy may be an issue for pumped systems with large-diameter pipes or boxes. The designers must consider this. For consistency, all discussion in the SWMP will refer to concrete pipes.

Figure 3-2. Proposed Improvements in Study Area 1A

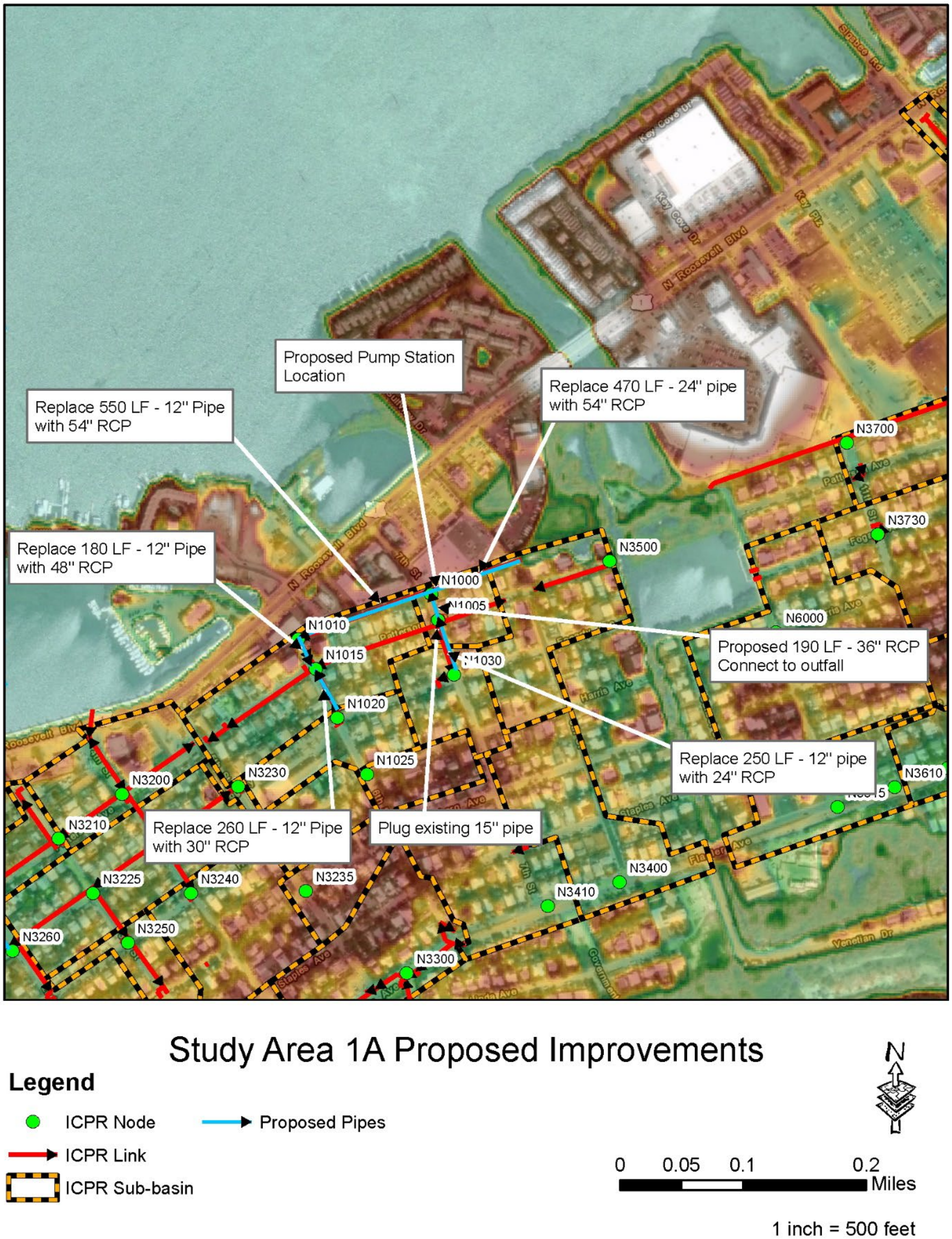


Table 3-1. Summary of Study Area 1A Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes

Study Area Sub-basins	10-year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/pump)
Patterson Avenue at 7th Street (N1005)	1.76	2.67	1.71	2.68	0.69
Fogarty Avenue at 7th Street (N1030)	1.23	2.12	1.86	2.38	1.07
Patterson Avenue at 6th Street (N1015)	1.14	2.09	1.71	2.37	0.70
Fogarty Avenue at 6th Street (N1020)	1.17	2.10	1.79	2.37	0.90

Because the elevations in this study area are slightly higher than some of the other study areas, there is a greater flood stage reduction when compared to the elevation 1.0-foot NAVD 88 tide boundary scenario. This may allow for selective road elevation raising in this area to remove flood stage from the road under these conditions. As the existing flooding is within 0.6 foot of the roadway low points and this is a local road, road raising has not been considered as a part of the preferred solution costs for this alternative.

In addition to the increased conveyance capacity headed to the outfall to the east, it is understood that the tidal salt pond this study area discharged to, located at the northern end of Sunset Drive, is environmentally sensitive and experiences sunny-day flooding on top of the conditions caused from typical rain events. The City is currently evaluating the Sunset Drive area with an ongoing project, including considerations for a tidal barrier that may alter flow from the west side. The pump station proposed for Study Area 1A may be modified to be located farther east, where it may direct the discharge flow away from the salt pond as needed to fit the two project needs. Study Area 1A also may be suitable for implementation of a modified gravity well or pump-assisted injection well concept as discussed in Section 4. Costs for additional injection wells are not included in the cost opinion for the near-term alternative.

Class 4 cost opinions were developed for the preferred solution both with and without pumps included. Table 3-2 identifies the estimated costs without a pump station in place (costs associated with gravity based- conveyance improvements only), while Table 3-3 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-2. Study Area 1A Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$2,152,549
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$322,882
Contractor Profit	10%	$215,255
Engineering/Design	22%	$473,561
Contingency/Market Volatility	25%	$672,672
Total Including Contingencies		$3,836,919

Table 3-3. Study Area 1A Cost Opinion – with Pump Station

Construction Subtotal (with Pump Station)		$6,275,075
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$941,261
Contractor Profit	10%	$627,508
Engineering/Design	22%	$1,380,517
Contingency/Market Volatility	25%	$1,960,961
Total Including Contingencies		$11,185,322

3.2.2 Study Area 1B, 3rd Street and Patterson Avenue

Study Area 1B lies in the central portion of the island, on the northwest side of New Town. Low-lying areas at frequently flooded intersections are at or near elevation 1.0 foot NAVD 88. Review of historical aerial photography of the area reveals frequent ponding and associated roadway damage along 3rd Street. The existing stormwater system includes inlets in higher-elevation areas to the south that convey flow into the study area where the system likely experiences a backflow condition during large storm events. The existing stormwater system also is undersized, with many pipes less than 12 inches in diameter. For purposes of this SWMP, this area was called the Patterson Avenue Improvement Zone.

The proposed conveyance pipe sizing and routing for this area includes bypassing existing pipe connections around the lower-lying areas to prevent any backflow from coming directly into those areas. Conceptual conveyance pipe sizing and routing based on current tidal boundary conditions is included on Figure 3-3. Proposed flood stage results are included in Table 3-4. The outfall pipe crossing FDOT-owned North Roosevelt Boulevard must be increased in size to allow for stormwater to discharge during both near-term and long-term tide boundary conditions.

The proposed improvements for the preferred alternative include the following:

- Plug existing 12-inch-diameter pipe headed north from the intersection of Harris Avenue at 3rd Street.
- Install proposed 350 LF of 30-inch-diameter elliptical reinforced concrete pipe (ERCP) from Harris Avenue at 3rd Street to Harris Avenue at 4th Street. ERCP is suggested given the low landscape but the final pipe material and size is the responsibility of the design.
- Remove 220 LF of 10-inch-diameter polyvinyl chloride pipe along 4th Street to the intersection with Fogarty Avenue and replace with 30-inch-diameter ERCP.
- Remove 275 LF of existing 15-inch-diameter pipe along 4th Steet from Fogarty Avenue to Patterson Avenue and replace with 42-inch-diameter ERCP. Connect 42-inch-diameter ERCP to proposed pump station vault proposed near the intersection of 4th Street at Patterson Avenue.
- Remove 280 LF of existing 8-inch-diameter pipe along 2nd Street from Harris Avenue to Fogarty Avenue; plug the existing connection east along Fogarty Avenue.
- Install proposed 530 LF of 30-inch-diameter ERCP along 2nd Street from Harris Avenue to Patterson Avenue.
- Remove 780 LF of existing 12-inch-diameter and 8-inch-diameter pipe along Patterson Avenue from 2nd Street to the proposed stormwater pump station vault located near the intersection with 4th Street and replace with 30-inch-diameter ERCP.
- Proposed 395 LF of 48-inch-diameter ERCP outfall pipe along 4th Street from Patterson Avenue to the outfall north of North Roosevelt Boulevard; existing 24-inch-diameter outfall pipe to remain.
- Proposed 150-cfs-peak-flow stormwater pump station located near the intersection of Patterson Avenue at 4th Street; proposed vault to be 440 feet, extending to elevation -10 feet NAVD 88; pump station to tie into proposed new outfall pipe for discharge piping.

Figure 3-3. Proposed Improvements in Study Area 1B

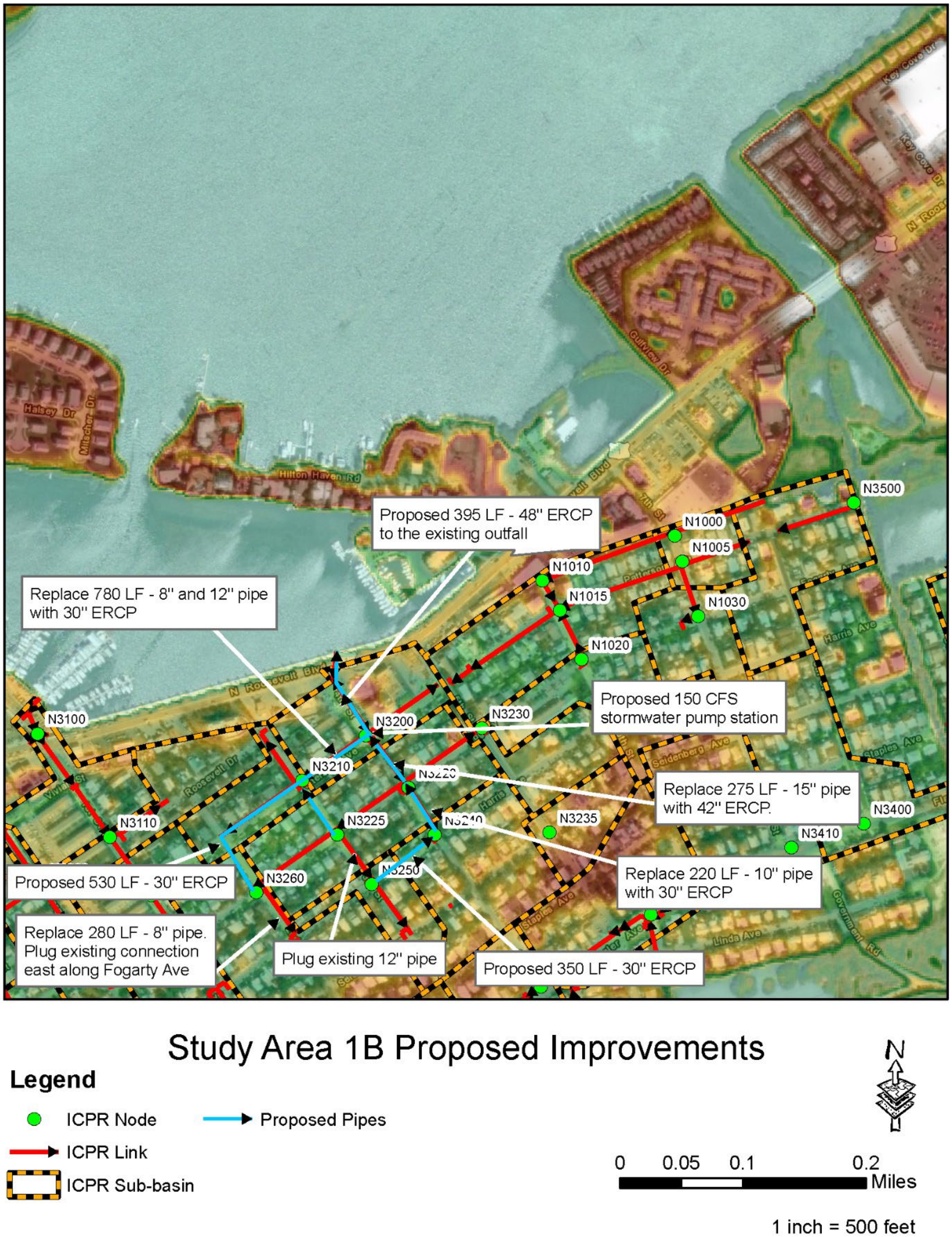


Table 3-4. Summary of Study Area 1B Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes

Study Area Sub-basins	10-Year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/pump)
Patterson Avenue at 2nd Street and 3rd Street (N3210)	1.12	2.06	1.82	2.37	0.00
Patterson Avenue at 4th Street (N3200)	0.61	2.02	1.74	2.37	0.00
Fogarty Avenue at 3rd Street and 4th Street (N3220)	0.79	2.08	1.84	2.37	0.00
Harris Avenue at 3rd Street (N3250)	1.34	2.08	1.86	2.37	0.69
Harris Avenue at 2nd Street (N3260)	1.07	2.08	1.86	2.37	0.15
Harris Avenue at 4th Street (N3240)	1.27	2.12	1.87	2.37	0.61

Because of the low elevations along the roadway and the total drainage area tributary to this outfall, conveyance pipe and stormwater pump station sizing in this area will be directly influenced by both the desired flood LOS for the area (whether any minor flooding is allowed during the subject storm event) and an ability to raise roadway elevations throughout the area. The DEM indicates that the low elevations continue through some of the residential parcels, creating a challenge for either option.

Class 4 cost opinions were developed for the preferred solution both with and without pumps. Table 3-5 identifies the estimated costs without a pump station in place (costs associated with gravity based- conveyance improvements only), while Table 3-6 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-5. Study Area 1B Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$4,895,860
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$734,379
Contractor Profit	10%	$489,586
Engineering/Design	22%	$1,077,089
Contingency/Market Volatility	25%	$1,529,956
Total Including Contingencies		$8,726,871

Table 3-6. Study Area 1B Cost Opinion – with Pump Station

Construction Subtotal (with Pump Station)		$13,140,912
Markups		
Contractors Overhead, General Conditions, Temp Facilities	15%	$1,971,137
Contractor Profit	10%	$1,314,091
Engineering/Design	22%	$2,891,001
Contingency/Market Volatility	25%	$4,106,535
Total Including Contingencies		$23,423,676

3.2.3 Study Areas 2A, 2B, and 2C, Tropical to George

Study Areas 2A, 2B, and 2C are in the middle of the island (on the east side of Old Town) and in close proximity to each other. These drainage sub-basins are essentially well connected through overland or channelized flow via roadways and intersections. There are higher elevations to the south before Flagler Road that create a ridge that naturally forces the stormwater from Study Area 2B north to the lower-lying roads of Study Area 2A. Study Area 2C is east of and adjacent to Study Area 2A. Analyses that included independent modeling simulations for Study Areas 2A and 2B did not reveal a solution that could isolate either sub-basin without raising roads or blocking other overland flow routes to create barriers. Altering topography was not considered feasible given the surrounding flat and densely developed residential property.

In general, the recommended conveyance strategy is to direct runoff to Jose Marti Pond because it is the closest outfall location. Given the existing piping leading there, this would require significant rework in some blocks in front of Horace O'Bryant School. This network is discussed later in this section. However, there is an important consideration that needs to be highlighted prior to discussing the entire proposal. The proposed outfall for this system is by the Jose Marti Drive area next to the pond (node Jose Marti Pond), which is also low-lying and currently experiences frequent flooding from rain events, as well as sunny-day flooding from King Tide events. The City is already planning a limited modification in the Jose Marti Drive area, so it was not included as a study area in the SWMP, but this is a key outfall location and worthy of a larger, regional project. This Jose Marti Drive area collects runoff from upland sub-basins to the west as well as those from the south and southeast.

Review of the flood-reduction benefits of proposed solutions versus existing tide conditions of 0 foot NAVD 88 and 1.0 foot NAVD 88 reveals that with an upgraded conveyance system and reductions in peak flood elevations are offset with even the modest near-term SLR. Consequently, to alleviate flooding conditions throughout this neighborhood, the overall area would benefit from a larger, regional pump station solution. Based on modeling several potential scenarios versus both near-term and long-term projected tidal boundary conditions, the following stormwater conveyance system upgrades are proposed:

- Proposed 415 LF of 24-inch-diameter RCP along Von Phister Street from Tropical Street to Leon Street; plug existing gravity wells through this area.
- Proposed 475 LF of 30-inch-diameter RCP along Leon Street from Von Phister Street to South Street; plug existing gravity wells through this area.
- Proposed 915 LF of 42-inch-diameter RCP along Leon Street from South Street to Catherine Street.
- Disconnect and plug existing 24-inch-diameter pipe connection headed north along Thompson Street at intersection with Seminary Street.
- Proposed 390 LF of 36-inch-diameter RCP along Seminary Street from Thompson Street to Leon Street.
- Proposed 675 LF of 36-inch-diameter RCP along Leon Street from Seminary Street to Catherine Street.
- Remove and dispose of 675 LF of existing 42- and 36-inch-diameter pipe along Leon Street from Duncan Street to Jose Marti Pond.
- Proposed 450 LF of 60-inch-diameter RCP along Leon Street from Catherine Street to proposed Bayview Park pump station vault.
- Proposed 100 LF of 36-inch-diameter RCP from Jose Marti Drive to Bayview Park pump station vault.
- Proposed-150-cfs-peak-flow stormwater pump station at Bayview Park; vault to be 440 feet, extending to elevation -10 feet NAVD 88.
- Proposed 100 LF of 60-inch-diameter pump discharge to Jose Marti Pond.

These upgrades are planned to be incorporated into a regional pump station located in the Bayview Park area near Jose Marti Drive. A map outlining the proposed improvements is included on Figure 3-4.

Figure 3-4. Proposed Improvements in Study Areas 2A, 2B, and 2C

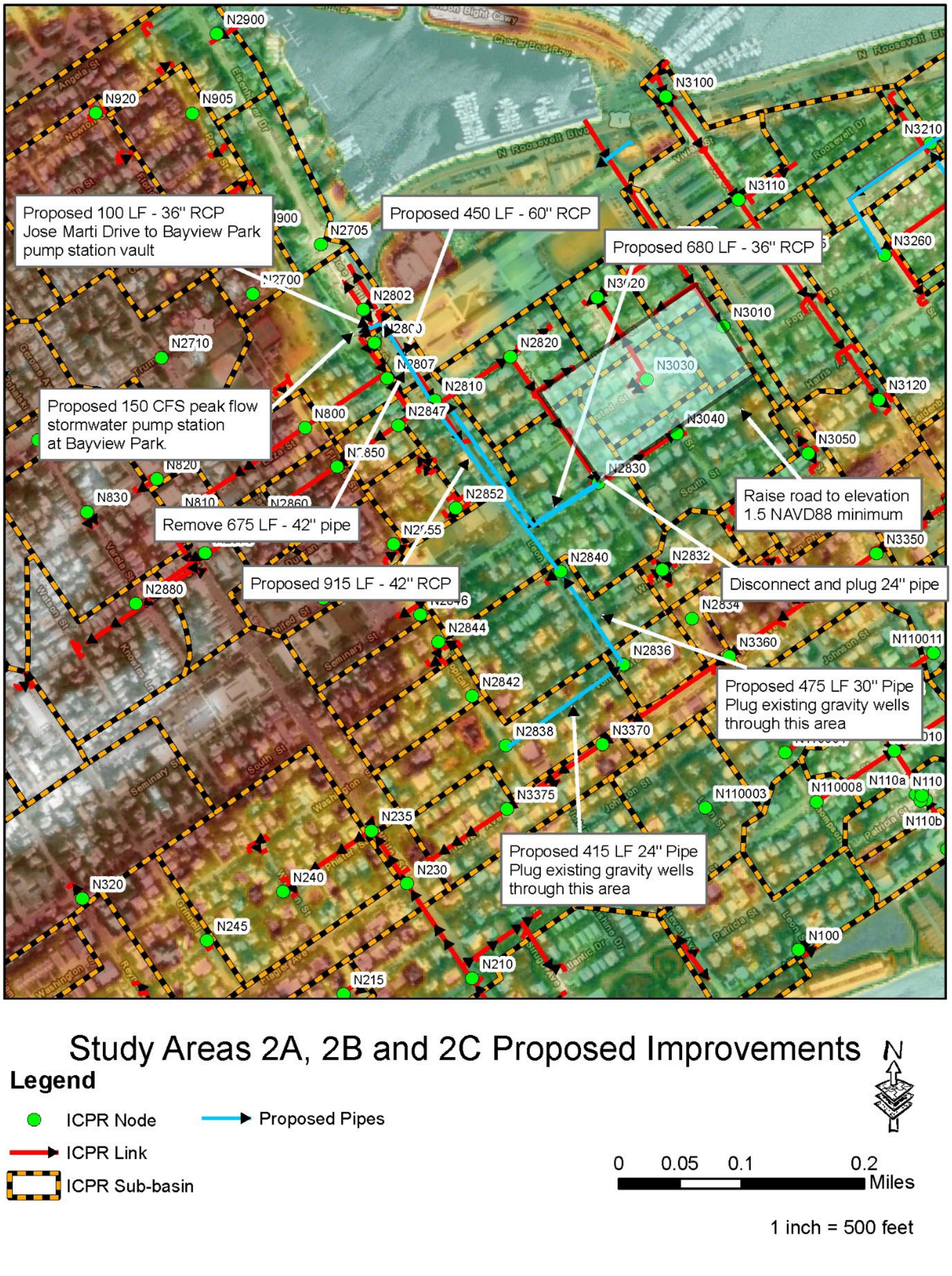


Table 3-7 provides the simulation results for the recommended conveyance system improvements with 1-foot boundary conditions as compared to the existing conditions and roadway low elevations. Table 3-7 also identifies existing conditions with a 2.7-foot boundary condition and a modification to the conveyance system that includes a regional stormwater pump station rated for 150 cfs (67,325 gallons per minute). The results table includes an incidental benefit to Study Area 2C upon installation of the pump station at Bayview Park.

Table 3-7. Summary of Study Areas 2A and 2B Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes and a Regional Pump Station

Study Area Sub-basins	10-Year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/pump)
Washington Street at Leon Street (N2840)	1.71	2.32	2.02	2.34	1.44
Seminary Street at Thompson Street (N2830)	1.53	2.12	2.02	2.31	1.97
Catherine Street at Leon Street (N2810)	1.66	1.74	1.34	2.31	0.28
Jose Marti Drive (N2802)	0.76	1.84	1.49	2.34	0.41
Von Phister Street at Tropical Street (N2838)	2.58	2.88	2.76	2.88	2.74
Von Phister Street at Leon Street (N2836)	2.47	2.68	2.26	2.68	1.89
Benefits Incidental to Study Area 2C					
United Street at Ashby Street (N3030)	1.06	2.10	N/A	2.31	1.91
Ashby Street at Seminary Street (N3040)	1.65	2.12	N/A	2.31	1.96
United Street at George Street (N3010)	1.30	1.84	N/A	2.31	1.98

Class 4 cost opinions have been developed for the preferred solution for Study Areas 2A and 2B, both with and without pumps. Table 3-8 identifies the estimated costs without a pump station in place (costs associated with gravity-based conveyance improvements only), while

Table 3-9 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-8. Study Areas 2A and 2B Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$4,592,780
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$688,917
Contractor Profit	10%	$459,278
Engineering/Design	22%	$1,010,412
Contingency/Market Volatility	25%	$1,435,244
Total Including Contingencies		$8,186,631

Table 3-9. Study Areas 2A and 2B Cost Opinion – with Pump Station

Construction Subtotal (no Pump Station)		$12,837,832
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$1,925,675
Contractor Profit	10%	$1,283,783
Engineering/Design	22%	$2,824,323
Contingency/Market Volatility	25%	$4,011,823
Total Including Contingencies		$22,883,436

As presented in Table 3-10, Study Area 2C near the intersection of United Street and Ashby Street sees an incidental benefit from the proposed improvements to Study Areas 2A and 2B to the west. Because there are recent stormwater projects constructed in this area, the preferred alternative includes leaving those improvements in place and raising road elevations at the low-lying intersection to a minimum 1.5 feet NAVD 88. This presents a solution that will prevent large-scale removal and replacement of recent construction work and limit flood depth and duration during the 10-year, 24-hour storm event.

Class 4 cost opinions have been developed for the preferred solution for Study Area 2C as shown in Table 3-10. The cost opinion for this solution is based on raising roadway elevations and adjusting utilities, curb and gutter, and sidewalk.

Table 3-10. Study Area 2C Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$1,117,808
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$167,671
Contractor Profit	10%	$111,781
Engineering/Design	22%	$245,918
Contingency/Market Volatility	25%	$349,315
Total Including Contingencies		$1,992,493

3.2.4 Study Area 3A, Riviera Drive

Study Area 3A includes Riviera Drive from 11th Street to the west and 17th Street to the east (western end). There are two small-diameter outfalls servicing Riviera Drive exclusively that discharge south to the Riviera Canal through residential properties that are closely spaced. There are also two larger-diameter outfalls that service both Flagler Avenue to the north and Riviera Drive. These outfalls are located through alleys that lead directly to the canal and are easily accessible for conveyance improvements. Riviera Drive elevations are lower through these areas, which contributes to frequent, sustained flooding through the area. This portion of Riviera Drive will become increasingly vulnerable to sunny-day flooding and will be overcome without a resiliency plan for the area that includes raising seawall elevations along both private and public seawalls in the area. These resiliency measures should be evaluated independently of this analysis. When a solution to raise the seawalls can be identified, the preferred solution for reducing flood stage in the area may be implemented.

Resilience-based solutions such as raising the elevation along Riviera Drive by repaving and improving drainage features will be limited, as they must prevent blocking flow from Flagler Avenue and contributing to flooding conditions along the county road. A review of the available elevation data identified that pavement elevations may be limited to 1.5 feet to 2.0 feet NAVD 88 without requiring additional improvements along Flagler Avenue. Road raising projects also will need to raise elevations along Kennedy Drive, Riviera Street, and 11th Street between the two roads.

The proposed solution for this area includes a combination of raising road elevations and improving outfall conveyance capacity of the larger two outfalls to the canal. The proposed solution also will require stormwater vaults and pump stations located at 11th Street and Riviera Street where the seawall elevation in the direct vicinity can be built up within the right of way.

The proposed improvements for the preferred alternative include the following:

- Install proposed 180 LF of 48-inch-diameter ERCP outfall from Riviera Drive to the canal at 11th Street; existing 42-inch outfall extending from Flagler Avenue to remain.
- Install proposed 180 LF of 48-inch-diameter ERCP outfall from Riviera Drive to the canal at Riviera Street; existing 48-inch-diameter outfall extending from Flagler Avenue to remain.
- Install tidal check valves at both the existing and proposed outfall locations.

- Raise existing roadway areas to elevation of 1.5 feet NAVD 88 at minimum or higher at Riviera Street, 11th Street, Kennedy Drive, and other low-lying areas along Riviera Drive; install curb and gutter system and roadside drainage along these areas to prevent blocking existing drainage paths.
- Proposed 50-cfs peak-flow stormwater pump stations located in the Riviera Street and 11th Street alleys south of Riviera Drive; proposed vault to be 440 feet extending to elevation -10 feet NAVD 88; pump station to tie into proposed new outfall pipe for discharge piping.

Figure 3-5 provides a schematic of the preferred conceptual solution for the area. Table **3-11** presents a summary of Study Area 3A simulation results.

Figure 3-5. Proposed Improvements in Study Area 3A

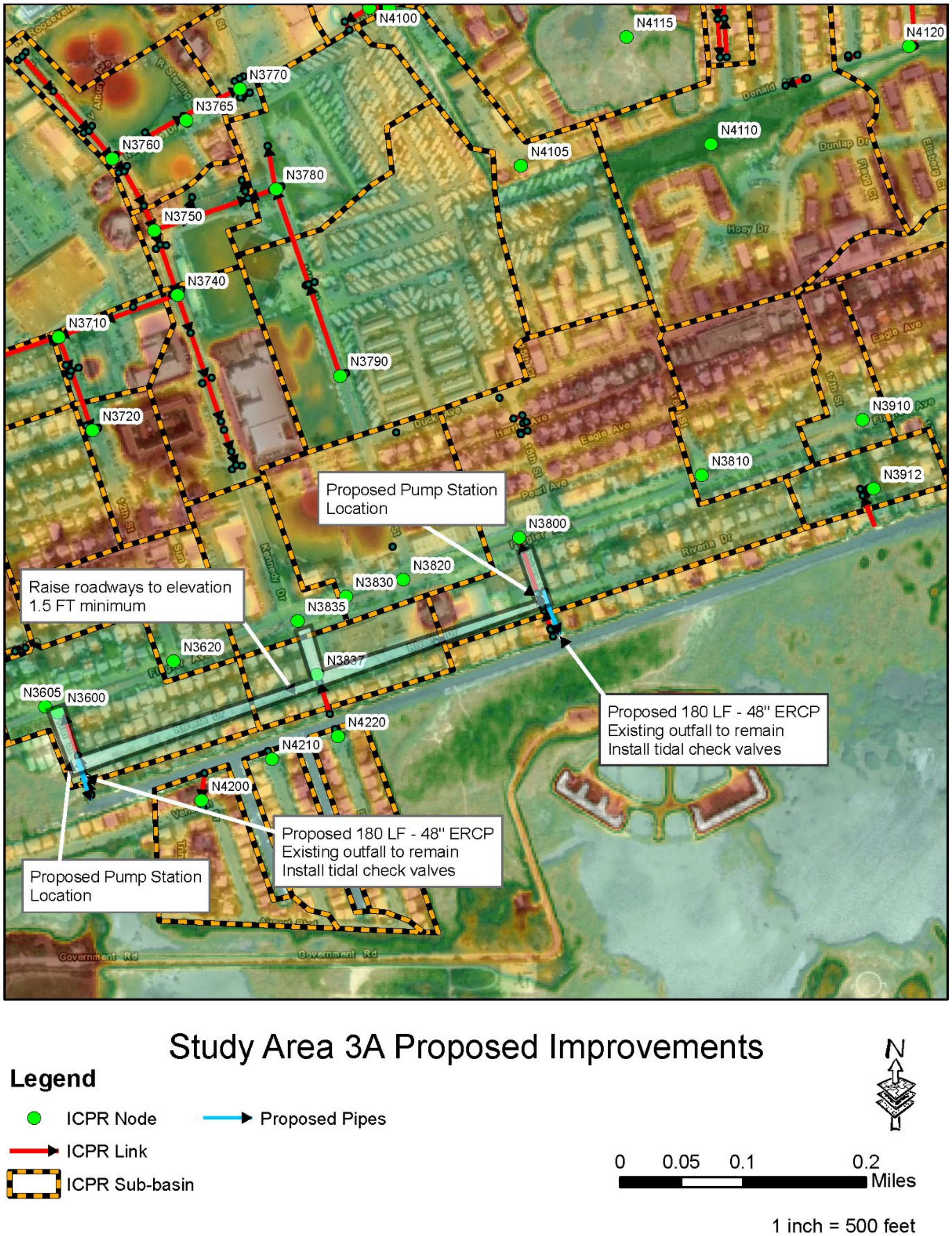


Table 3-11. Summary of Study Area 3A Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes and a Pump Station

Study Area Sub-basins	10-Year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/pumps)
11th Street at Riviera Drive (N3600)	0.58	1.93	1.33	2.70	1.04
Kennedy Drive at Riviera Drive (N3837)	0.65	2.09	2.01	2.71	2.02
Riviera Street at Riviera Drive (N3800)	0.65	1.99	1.43	2.71	1.00
17th Street at Riviera Drive (N3912)	1.46	2.39	2.39	2.77	2.39

The preferred solution makes use of the existing larger outfalls from Flagler Avenue and services the area with its own expanded outfalls that handle both Riviera Drive and overflow from flooding on Flagler Avenue. When paired with the road raising included with this solution, flooding during the 10-year, 24-hour event will be limited to within 6 inches for a limited duration when subjected to the 1.0-foot NAVD 88 tide boundary condition. Road raising also will reduce flood depth and duration for the tow areas along Riviera Drive currently serviced by outfalls that are not easily accessible as they pass through private property leading to the canal. Directly connecting these areas to the new outfalls does not appear to provide a suitable reduction in flood stage and duration by comparison.

Class 4 cost opinions were developed for the preferred solution both with and without pumps. Table 3-12 identifies the estimated costs without a pump station in place (costs associated with gravity-based conveyance improvements only), while Table 3-13 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-12. Study Area 3A Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$2,415,964
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$362,395
Contractor Profit	10%	$241,596
Engineering/Design	22%	$531,512
Contingency/Market Volatility	25%	$754,989
Total Including Contingencies		$4,306,455

Table 3-13. Study Area 3A Cost Opinion – with Pump Station

Construction Subtotal (with Pump Station)		$10,035,137
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$1,505,270
Contractor Profit	10%	$1,003,514
Engineering/Design	22%	$2,207,730
Contingency/Market Volatility	25%	$3,135,980
Total Including Contingencies		$17,887,631

3.2.5 Study Areas 4A and 4B, East End

Study Areas 4A and 4B are within the northern portion of the East End Resiliency Opportunity Zone and defined generally as the residential areas around Donald Avenue and 20th Street, east to Flagler Avenue. This entire East End has low slopes and the existing stormwater piping does not provide sufficient drainage capacity. The area has a low-elevation landscape and is vulnerable to SLR. Significant new conveyance will be required to move water to larger outfalls.

While these two areas are independent sub-basin areas, analysis of the potential solutions identified for each showed increased benefit from simulating the proposed solutions as a group because Study Area 4A includes undersized conveyance piping that shares an outfall with an already restricted 42-inch-diameter pipe from the Flagler Avenue system. The basins have similar landscape elevations, which increases overland sheet flow between them, especially during future tidal boundary elevation scenarios.

In addition to the restricted conveyance, study area elevations are lower through these areas, which contributes to frequent, sustained flooding. The eastern area of the City will become increasingly vulnerable to sunny-day flooding and will be overcome without a resiliency plan for the area that includes raising seawall elevations along both private and public seawalls or other solutions. These resiliency measures should be evaluated independently of this analysis. When a solution to raise the seawalls can be identified, the preferred solution for reducing flood stage in the area may be implemented.

The preferred solution alternative for Study Area 4A includes the following proposed improvements:

- Proposed 955 LF of 42-inch-diameter ERCP along 20th Street from Duck Avenue to Flagler Avenue.
- Proposed 685 LF of 42-inch-diameter ERCP along Flagler Avenue from 20th Street to 19th Street.
- Proposed 175 LF of 54-inch-diameter ERCP along 19th Street from Flagler Avenue to Sunrise Lane, to the canal along Sunrise Lane.
- Existing stormwater network through roadways to remain and tie to new proposed pipes and inlets.
- Proposed 150-cfs peak-flow stormwater pump station near 19th Street at Sunrise Lane; consider converting the road in this area to an alleyway or otherwise modifying it to fit; vault to be 440 feet extending to elevation -10 feet NAVD 88.

The preferred solution alternative for Study Area 4B includes the following proposed improvements:

- Remove 300 LF of existing 12-inch-diameter pipe along 20th Street from Paula Avenue to Donald Avenue and replace with 48-inch-diameter RCP.
- Remove 400 LF of existing 15-inch-diameter pipe along Donald Avenue from 19th Street to 20th Street and replace with 48-inch-diameter RCP.
- Remove 600 LF of existing 36-inch-diameter pipe along 20th Street from Donald Avenue to the outfall pipe near 24 North Hotel and replace with 48-inch-diameter RCP.
- Remove 750 LF of existing 36-inch-diameter pipe along outfall near 24 North Hotel and replace with twin 48-inch-diameter RCP.
- Proposed 150-cfs peak-flow stormwater pump station near 20th Street at the outfall pipe; vault to be 440 feet extending to elevation -10 feet NAVD 88.

Figure 3-6 presents the overall schematic associated with the preferred alternative proposed improvements in these areas.

Figure 3-6. Proposed Improvements in Study Areas 4A and 4B

Several alternatives were evaluated for this area that focus on routing the conveyance through shared outfalls or oversizing the conveyance system to either eliminate roadway flooding for the 10-year, 24-hour event or reduce the peak stage to limit the anticipated inundation to less than 0.25 foot above the lower roadway segments. These alternatives proved that, although possible, there is a diminishing return on value in oversizing the pipes to the level required. Likewise, the area will continue to be subjected to SLR that will further degrade the expected peak flood stage reduction without the use of a pump station. Table 3-14 presents the flood stage results from simulations of Study Areas 4A and 4B.

Table 3-14. Summary of Study Areas 4A and 4B Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes and Pump Stations

Study Area Sub-basins	10-year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/ pump)
20th Street East to Flagler Avenue (N3930)	1.75	2.71	2.16	2.90	0.22
Flagler Avenue Northeast of 17th Street (N3900)	1.52	2.28	1.40	2.77	0.00
Northside Drive (N4125)	2.30	2.58	1.80	2.88	0.00
20th Street near Donald Avenue (N4130)	1.65	2.71	2.16	2.90	0.15
Donald Avenue near 20th Terrace (N4147)	1.31	2.85	2.34	2.94	0.86
20th Street from Paula Avenue to Cindy Avenue (N4150)	1.69	2.71	2.14	2.90	0.21
Northside Drive near 18th Terrace (N4160)	2.59	2.72	2.28	2.89	0.00

Class 4 cost opinions were developed for the preferred solution for Study Areas 4A and 4B, both with and without pumps. Table 3-15 and Table 3-16 identify the estimated costs without a pump station in place (costs associated with gravity-based conveyance improvements only), while Table 3-17 and Table 3-18 identify the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-15. Study Area 4A Cost Opinion – without Pump Station

Study Area 4A Construction Subtotal (no Pump Station)		$2,681,795
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$402,269
Contractor Profit	10%	$268,179
Engineering/Design	22%	$589,995
Contingency/Market Volatility	25%	$838,061
Total Including Contingencies		$4,780,299

Table 3-16. Study Area 4A Cost Opinion – with Pump Station

Study Area 4A Construction Subtotal (with Pump Station)		$10,926,847
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$1,639,027
Contractor Profit	10%	$1,092,685
Engineering/Design	22%	$2,403,906
Contingency/Market Volatility	25%	$3,414,640
Total Including Contingencies		$19,477,104

Table 3-17. Study Area 4B Cost Opinion – without Pump Station

Study Area 4B Construction Subtotal (no Pump Station)		$3,760,106
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$564,016
Contractor Profit	10%	$376,011
Engineering/ Design	22%	$827,223
Contingency /Market Volatility	25%	$1,175,033
Total Including Contingencies		$6,702,389

Table 3-18. Study Area 4B Cost Estimate – with Pump Station

Study Area 4B Construction Subtotal (with Pump Station)		$12,005,158
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$1,800,774
Contractor Profit	10%	$1,200,516
Engineering/Design	22%	$2,641,135
Contingency/Market Volatility	25%	$3,751,612
Total Including Contingencies		$21,399,194

3.2.6 Study Area 4C, Kennedy Drive to 15th Court

Study Area 4C includes a large drainage area located from Kennedy Drive to 15th Court in the middle of New Town. Two outfalls service the area, including a 60-inch-diameter pipe extending west from Kennedy Drive that discharges west of 10th Street and an open channel extending north along the eastern side of the mobile home park, where flow becomes restricted upon crossing Northside Drive. Study Area 4D also makes use of the open channel outfall because of the similar elevations across the area and overland flow connectivity from the contributing basins.

The preferred solution alternative for Study Area 4C includes the following proposed improvements:

- Plug existing pipe connection headed north on Glynn Archer Jr. Drive just south of Rex Weech Field and install new inlets and proposed 545 LF of 36-inch-diameter ERCP west to Kennedy Drive.
- Remove and replace 230 LF of existing 24-inch-diameter pipe with 36-inch-diameter ERCP north along Kennedy Drive to the west main outfall.
- Remove and replace 260 LF of existing 24-inch-diameter pipe with 30-inch-diameter ERCP from Glynn Archer Jr. Drive just north of Rex Weech Field.
- Remove and replace the 290 LF of existing 30-inch-diameter pipe with 48-inch-diameter ERCP south along Kennedy Drive to the west main outfall.
- Remove and replace the 360 LF of existing 24-inch-diameter pipe with 36-inch-diameter RCP from Kennedy Drive at the Northside Drive intersection to the west main outfall.
- Connect the existing 60-inch-diameter west main outfall to a proposed 150-cfs peak-flow stormwater pump station located near the commercial area just north of the Patterson Avenue at 10th Street intersection; vault to be 440 feet extending to elevation -10 feet NAVD 88.
- Install 250 LF of discharge piping from the pump station to the outfall west of 10th Street.
- Selective road raising at low-lying areas of Glynn Archer Jr. Drive to minimum elevation of 1.5 feet NAVD 88.

Conceptual stormwater conveyance pipe sizing and routing for Study Area 4C based on current tidal boundary conditions is included on Figure 3-7.

Figure 3-7. Proposed Improvements in Study Area 4C

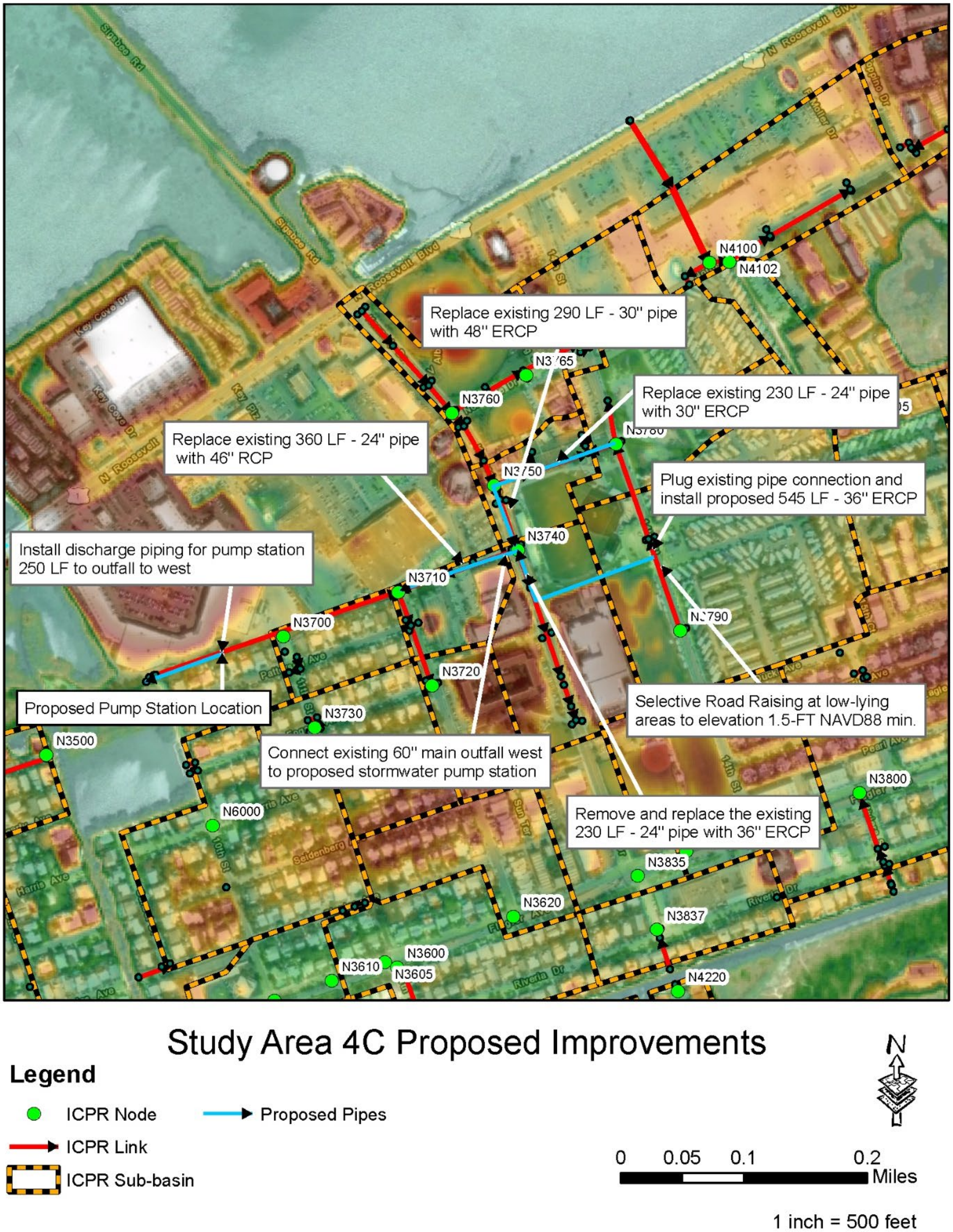


Table 3-19 provides the simulation results for the recommended conveyance system improvements with 1.0-foot boundary conditions as compared to the existing conditions and roadway low elevations. The table also identifies existing conditions with a 2.7-foot boundary condition and a modification to the conveyance system that includes the regional pump station.

Table 3-19. Summary of Study Area 4C Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes and Pump Stations

Study Area Sub-basins	10-year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/pump)
Glynn R. Archer Jr. Drive at Poinciana Field (N3790)	1.06	2.52	2.35	2.76	1.99
Glynn R. Archer Jr. Drive at Rex Weech Field (N3780)	1.24	2.34	2.27	2.75	1.64
Kennedy Drive at Northside Drive (N3760)	0.82	2.52	2.36	2.78	1.91
Kennedy Drive at Weech Field (N3750)	1.24	2.31	2.23	2.74	1.46
Kennedy Drive at Poinciana Field (N3740)	1.91	2.22	2.21	2.74	1.36
Patterson Avenue at 12th Street (N3710)	1.05	2.11	2.08	2.73	0.37
Patterson Avenue at 11th Street (N3700)	1.25	1.86	1.83	2.72	0.00

Review of Figure 3-7 indicates that the sub-basins along Glynn R. Archer Jr. Drive are unable to be fully removed from flooding during the 10-year, 24-hour storm event with the proposed improvements in place when subject to a tidal boundary condition of 2.7 feet NAVD 88. To fully remove the subject areas from the 10-year, 24-hour flood event, the proposed improvements include raising the road elevations where possible because the roadway locations below the flood stage are located in isolated areas where the opportunity is available. A review of the elevation information included in the DEM identifies that a minimum road elevation of 1.5 feet NAVD 88 may be suitable to prevent flooding in the localized areas by more than 6 inches when subject to the 10-year, 24-hour storm event.

It is important to note that the west outfall location presents an additional challenge when compared to the tidal boundary condition of 2.7 feet NAVD 88. The existing roads in the vicinity of the outfall are currently lower than the future tide boundary condition. These areas should be addressed independently as a part of meeting the resiliency standards outlined in the resiliency TM (Jacobs 2021b).

In addition to the increased conveyance capacity headed to the outfall to the west, it is understood that the small salt pond located at the northern end of Sunset Drive is environmentally sensitive and experiences sunny-day flooding on top of the conditions caused from typical rain events. The City is currently evaluating this area with a different project, including considerations for a tidal barrier that may push the increased flow to the area to the coast. In addition to the ongoing evaluation, the pump station proposed for this area may be modified to be located farther west where it may direct the discharge flow north of any tidal barrier or other features implemented as a part of that study. Costs for this are not included in the cost opinion for this alternative.

An alternative solution that directs flow to the north through a new outfall proposed through the Kennedy Drive at North Roosevelt Boulevard intersection is evaluated with Study Area 4D. The additional pump and outfall proposed with Study Area 4D provide a mutual benefit to the area that may prevent the Glynn R. Archer Jr. Drive area from flooding. As that solution includes a new outfall location, it may only be considered upon the decision to move forward with the solution proposed in Study Area 4D.

Class 4 cost opinions have been developed for the preferred solution both with and without pumps. Table 3-20 identifies the estimated costs without a pump station in place (costs associated with gravity-based conveyance improvements only), while Table 3-21 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-20. Study Area 4C Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$2,849,691
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$427,454
Contractor Profit	10%	$284,969
Engineering/Design	22%	$626,932
Contingency/Market Volatility	25%	$890,528
Total Including Contingencies		$5,079,574

Table 3-21. Study Area 4C Cost Opinion – with Pump Station

Construction Subtotal (with Pump Station)		$11,094,743
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$1,664,211
Contractor Profit	10%	$1,109,474
Engineering/Design	22%	$2,440,843
Contingency/Market Volatility	25%	$3,467,107
Total Including Contingencies		$19,776,379

3.2.7 Study Area 4D, Donald Avenue

Study Area 4D is located along the north and south sides of Donald Avenue and in the East End Resilience Opportunity Zone. Most of the area drains south to an open ditch and wetland on the south side of Donald Avenue. This ditch currently is a major flow path from drainage areas to the east (including Study Areas 4A and 4B). The Poinciana Plaza apartments to the south drain to this system as well.

This existing ditch drains west through an enclosed 72-inch-diameter arch pipe, then through an open channel that heads north to Northside Drive, where it travels underground beneath the commercial property (shopping center with Publix) and Roosevelt Boulevard through two 24-inch-diameter pipes that discharge to the coast. The conveyance capacity through the twin 24-inch-diameter pipes in restricted as compared to the open channel. The area along Donald Avenue is at or near elevation 1.0 foot NAVD 88, and the already limited storage capacity of the ponded area will be even more restricted when modeled with future tidal boundary conditions.

Although there is a stormwater system servicing some of the properties north of Donald Avenue, the existing pipe capacity is limited, as the low elevations throughout appear to encounter backflow conditions. The neighborhood also is serviced by a gravity well system that will experience reduced performance as SLR continues to impact the island.

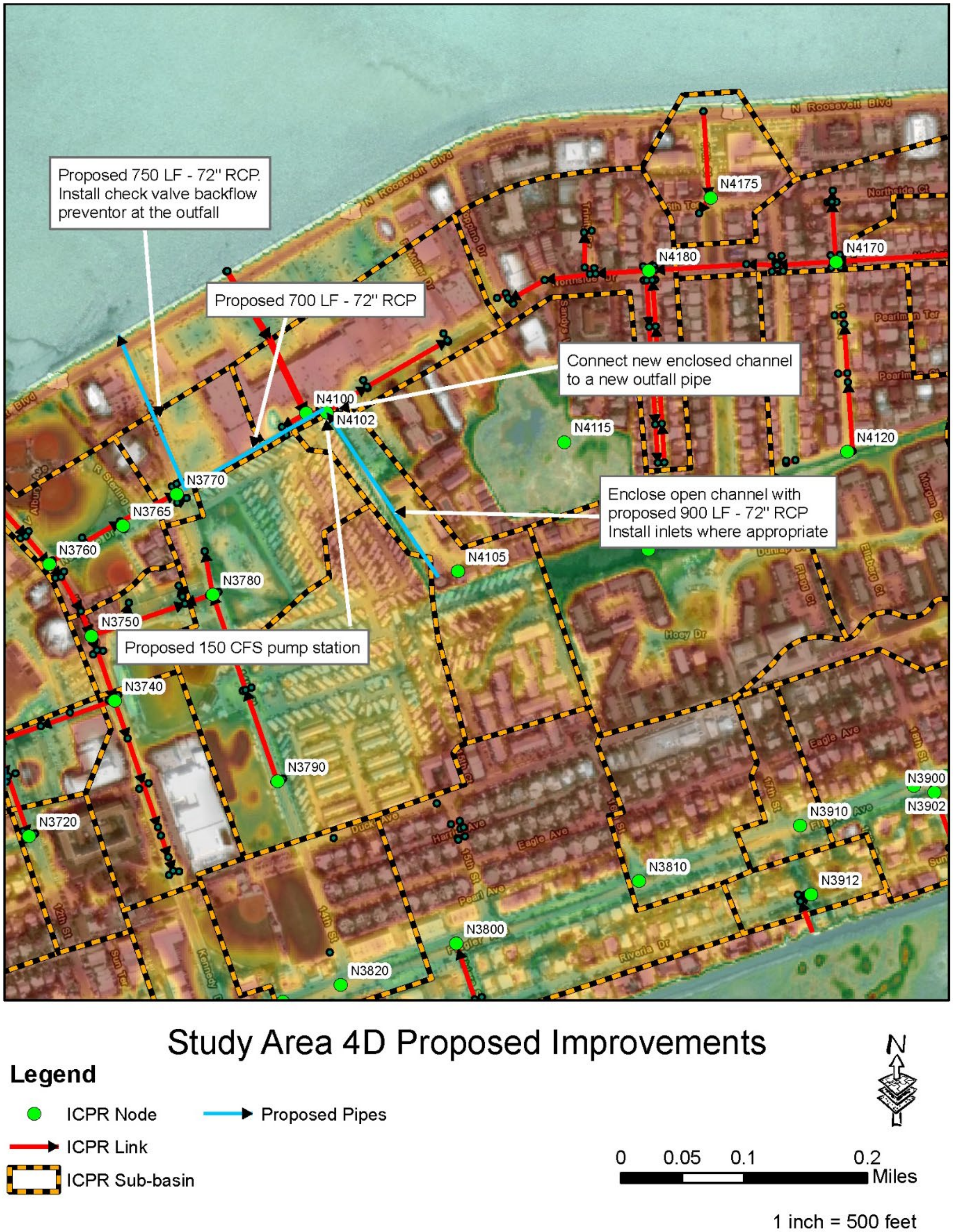
Several alternatives were evaluated for this area, with a focus on interconnecting the Donald Avenue stormwater system to either an additional proposed outfall location to the north or increasing outfall capacity through the Northside Drive system. Analysis also was completed to identify any potential to increase storage near the pond south of Donald Avenue by building up a berm or gravity wall without contributing to peak flood stage increases to the contributing area. The preferred alternative for this study area includes the following proposed improvements:

- Enclose the open channel headed north to Northside Drive with a proposed 900 LF of 72-inch-diameter RCP, maintaining a similar capacity as the previous project enclosing this channel as it heads east to Donald Avenue; install inlets along this pipe run where appropriate to maintain drainage from the surrounding area.
- Connect the new, enclosed channel system at Northside Drive to a new outfall pipe; allow the existing two 24-inch-diameter pipes to handle the outfall for the roadside system only at Northside Drive.

- Install proposed 700 LF of 72-inch-diameter RCP west along Northside Drive to the intersection with Glynn Archer Jr. Drive.
- Install proposed 750 LF of 72-inch-diameter RCP north to the outfall past North Roosevelt Boulevard; install a check valve backflow preventor at the outfall.
- Connect the proposed 72-inch-diameter outfall to a proposed 150-cfs peak-flow stormwater pump station located near the connection at Northside Drive; proposed vault to be 440 feet extending to elevation -10 feet NAVD 88.

Proposed new outfalls for Study Area 4D are shown on Figure 3-8.

Figure 3-8. Potential New Outfalls for Study Area 4D



The proposed improvements for this study area also are reflected in reduced flood stages for Study Area 4D, as shown in Table 3-22.

Table 3-22. Summary of Study Area 4D Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes and Pump Stations

Study Area Sub-basins	10-Year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/pump)
Donald Avenue West (N4110)	1.14	2.69	2.24	2.90	1.26
Donald Avenue at Poinciana Mobile Home Park (N4105)	1.38	2.68	2.19	2.90	1.01
16th Terrace at Donald Avenue (N4180)	2.10	2.69	2.35	2.90	2.37
Donald Avenue at 18th Street (N4120)	1.51	2.69	2.25	2.90	1.42
Donald Avenue at 19th Street (N4140)	2.02	2.75	2.53	2.91	2.22
Donald Avenue at 16th Terrace (N4147)	1.22	2.84	2.77	2.95	2.71

The proposed condition results identify that constructing and oversizing a new outfall that services the Donald Avenue area will provide a benefit to all lower-lying areas along the eastern part of the City. When in place, other study area solution alternatives may be modified to take advantage of the additional stormwater conveyance capacity through this area. For example, Donald Avenue at 19th Street and Donald Avenue at 16th Terrace could both be modified with larger conveyance pipes into the Donald Avenue area to remove their subject basins from the 10-year, 24-hour flood stage.

Class 4 cost opinions were developed for the preferred solution both with and without pumps. Table 3-23 identifies the estimated costs without a pump station in place (costs associated with gravity-based conveyance improvements only), while Table 3-24 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-23. Study Area 4D Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$3,529,222
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$529,383.31
Contractor Profit	10%	$352,922.21
Engineering/Design	22%	$776,428.86
Contingency/Market Volatility	25%	$1,102,881.90
Total Including Contingencies		$6,290,838

Table 3-24. Study Area 4D Cost Opinion – with Pump Station

Construction Subtotal (with Pump Station)		$11,774,274
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$1,766,141.09
Contractor Profit	10%	$1,177,427.39
Engineering/Design	22%	$2,590,340.26
Contingency/Market Volatility	25%	$3,679,460.60
Total Including Contingencies		$20,987,643

If there are water quality or other permitting complications associated with constructing the proposed new outfall, the area south of Donald Avenue also may be suitable for modified gravity injection wells or pump-assisted injection wells. The modified gravity injection well concept is described further in Section 3.3.2 of this report. These concepts may be ideal, as they will help address water quality issues at a larger scale based on the large drainage area contributory to the Donald Avenue ditch system. Likewise, the proposed regional pump station location near Northside Drive could be modified to use pump-assisted injection for smaller, more-frequent storm events, with the proposed outfall and peak pump rating used to service the area during larger storm events.

3.2.8 Study Area 5A, White Street to Grinnell Street

Study Area 5A is in the northwestern area of the island and is isolated from the other study areas. This area is in the White to Grinnell Neighborhood Drainage Zone. The neighborhood is currently serviced by a gravity well system that will experience reduced capacity as SLR continues to impact the island. The busy intersection of Eaton Street and White Street is at or near elevation 1.0 foot NAVD 88, where it receives incoming flow from the surrounding area with limited capacity to convey it to the outfall located at Grinnell Street (by the ferry terminal). The flow path between White Street and Grinnell Street is under commercial buildings and cannot be fully investigated. It includes some large, old pipes or culverts, but the flow path is constrained prior to the outfall. A lot of drainage areas south of the ferry use these outfalls. Because the near-term tide condition of 1.0 foot is already at the intersection's elevation, solutions for this area will require resiliency measures such as stormwater pumps, pump-assisted injection wells, road raising, or other regionalized solutions discussed in Section 4 of this report.

Conceptual stormwater conveyance pipe sizing and routing based on current tidal boundary conditions is included on Figure 3-7. The preferred gravity outfall route for the stormwater system is to tie directly to the 4-foot by 6-foot box culvert located north of the intersection on White Street. A proposed pump station may be located anywhere from the White Street at Eaton Street intersection to the outfall corridor down Grinnell Street or adjacent to it. The pump station will require an outfall with which to connect.

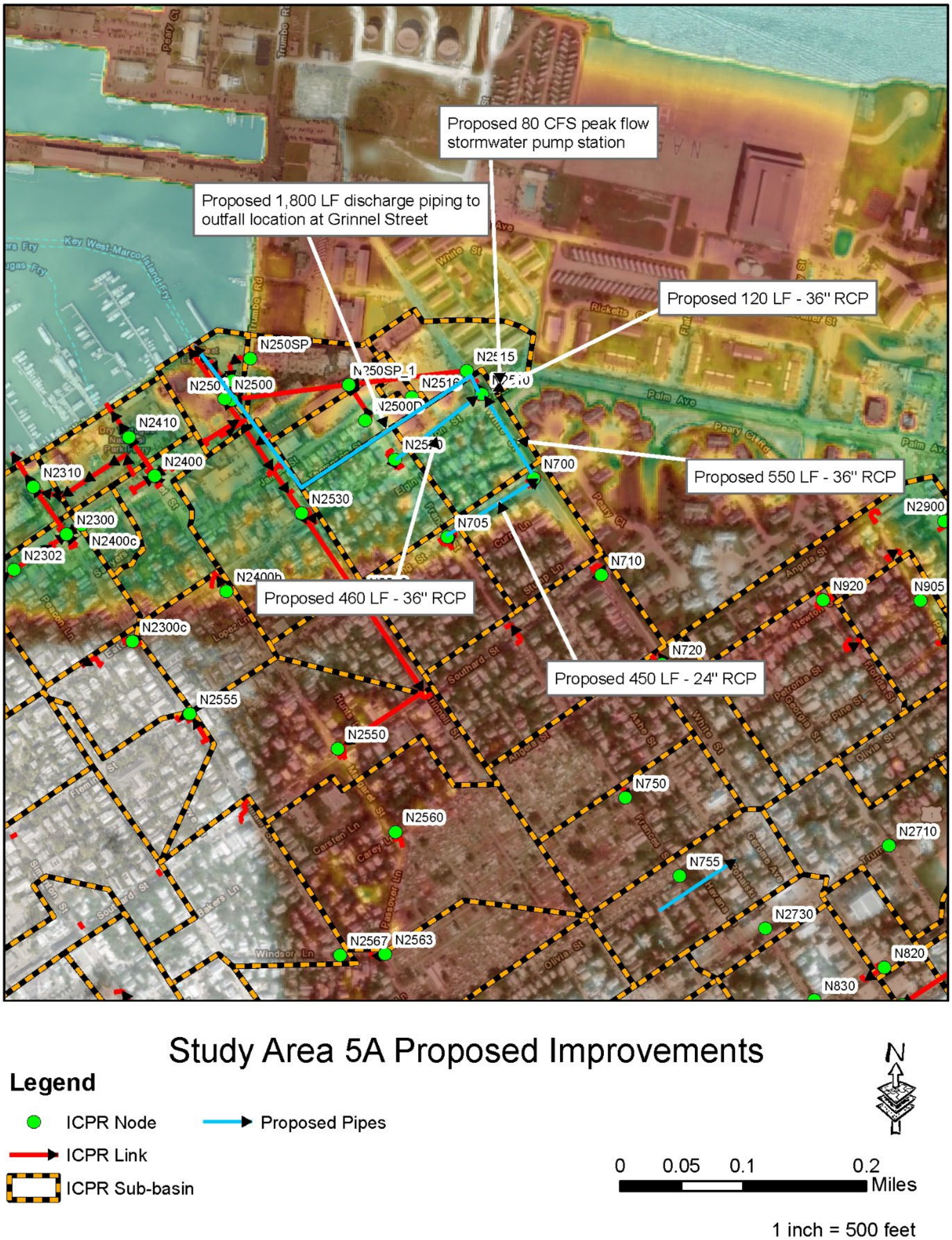
There is currently a vault near the ferry that is near the terminus of the culvert. The piping along Mustin Street to the Grinnell Street outfalls is not well understood. There may be a better way to connect this box culvert to an outfall. Increased pump vault capacity may help reduce stormwater pump station size requirements to provide a benefit for long-term future tide boundary conditions. Alternatively, a pump station and new outfall down White Street may be pursued, but this will cross U.S. Navy/federal property. The pump station discharge piping is estimated along the longer path to the outfalls to allow for optimization of the conveyance improvements when survey is obtained and the existing conditions are further analyzed.

The proposed improvements for the preferred alternative include the following:

- Proposed 450 LF of 24-inch-diameter RCP from Southard Street at Frances Street to the intersection of Fleming Street at White Street.
- Proposed 550 LF of 36-inch-diameter RCP from Southard Street at White Street to the proposed pump station vault near the intersection of White Street at Eaton Street.
- Proposed 460 LF of 36-inch-diameter RCP from Fleming Street at Frances Street to the intersection of Fleming Street at White Street.
- Proposed 120 LF of 36-inch-diameter RCP from Fleming Street at White Street to the proposed pump station vault near the intersection of White Street at Eaton Street.
- Proposed 80-cfs peak-flow stormwater pump station located near the intersection of White Street and Eaton Street; proposed vault to be 440 feet extending to elevation -10 feet NAVD 88.
- Proposed discharge piping to outfall location near Grinnell Street (1,800 LF). There are already two pipe outfalls here, so a new outfall may need to be east of the ferry terminal.

Figure 3-9 shows the proposed improvements for Study Area 5A.

Figure 3-9. Proposed Improvements in Study Area 5A



The proposed reduction in flood stage with the preferred solution in place is included in Table 3-25.

Table 3-25. Summary of Study Area 5A Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes and Pump Stations

Study Area Sub-basins	10-year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-ft Tide	Proposed Solution, 1-ft Tide (no pump)	Existing Conditions, 2.7-ft Tide	Proposed Solution, 2.7-ft Tide (w/pump)
Eaton Street at White Street (N2510)	1.19	2.02	1.70	2.48	0.00
Fleming Street at White Street (N700)	1.69	2.65	2.06	2.63	0.63
Fleming Street at Frances Street (N705)	2.34	3.19	2.94	3.20	2.22
Eaton Street at Frances Street (N2520)	1.11	2.04	1.87	2.48	0.73
Caroline Street at Grinnell Street (2500)	1.40	1.48	1.48	2.48	1.48

Class 4 cost opinions have been developed for the preferred solution both with and without pumps. Table 3-26 identifies the estimated costs without a pump station in place (costs associated with gravity-based conveyance improvements only), while Table 3-27 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-26. Study Area 5A Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$3,950,007
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$592,501
Contractor Profit	10%	$395,001
Engineering/Design	22%	$869,002
Contingency/Market Volatility	25%	$1,234,377
Total Including Contingencies		$7,040,888

Table 3-27. Study Area 5A Cost Opinion – with Pump Station

Construction Subtotal (with Pump Station)		$10,133,796
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$1,520,069
Contractor Profit	10%	$1,013,380
Engineering/Design	22%	$2,229,435
Contingency/Market Volatility	25%	$3,166,811
Total Including Contingencies		$18,063,492

3.2.9 Study Area 6A, Southern Bahama Village

Study Area 6A lies in the southwest corner of the island, just before the Naval Air Station. The study area focuses on flooding at the Frederick Douglass Gym, at the intersection of Emma and Olivia Streets, and at Nelson English Park, on the corner of Thomas and Catherine Streets. This study area represents all drainage infrastructure south of Geraldine Street and west of Thomas Street. Much of the area is low lying near the coast, with a majority of the existing infrastructure at an elevation of 4 feet NAVD 88 or less. There are currently no pump stations in the area, but there is a good-sized outfall flowing southeast on Fort Street out to the ocean. The system lacks connectivity to this outfall, resulting in sheet flow channelization in the streets and sustained flooding at the intersections. The system is overwhelmed and cannot direct flow from this area fast enough. However, near Nelson English Park, the streets are low (less than elevation 2 feet NAVD 88) and there is a sanitary pump station (Pump Station A) that has to be considered.

The proposed solution for this area includes adding connectivity to the intersection of Emma Street and Olivia Street, adding redundancy on Fort Street and Amelia Street, and installing two pump stations. In addition, a check valve backflow preventer is recommended at the outfall to reduce tidal influences at the existing outfall. Conceptual conveyance pipe sizing and routing based on current tidal boundary conditions is included on Figure 3-10. Proposed flood stage results are included in Table 3-28.

The proposed improvements for the preferred alternative include the following:

- Install proposed 330 LF of 36-inch-diameter RCP along Olivia Street from Emma Street to Fort Street.
- Install proposed 1,030 LF of 48-inch-diameter RCP along Fort Street from Olivia Street to Amelia Street, paralleling the existing 24-inch-diameter RCP.
- Install proposed 470 LF of 24-inch-diameter RCP along Thomas Street from Catherine Street to Amelia Street and continuing west along Amelia Street from Thomas Street to Howe Street, paralleling the existing 12-inch-diameter RCP.
- Install proposed 615 LF of 36-inch-diameter RCP along Amelia Street from Howe Street to Fort Street, paralleling the existing 24-inch-diameter and 36-inch-diameter RCP.
- Remove and replace 550 LF of 48-inch-diameter RCP from Fort Street to the outfall with 72--inch -diameter RCP.
- Install a tidal check valve at the existing outfall location. The City was already planning to do this.

- Construct proposed 50-cfs peak-flow stormwater pump station located in the parking lot at the Frederick Douglass Gym on the corner of Fort Street and Olivia Street; proposed vault extending to elevation -10 feet NAVD 88.
- Construct proposed 18.5-cfs peak-flow stormwater pump station located in the parking lot at the Nelson English Park; proposed vault extending to elevation -10 feet NAVD 88.

Figure 3-10 shows the proposed improvements for Study Area 6A.

Figure 3-10. Proposed Improvements in Study Area 6A

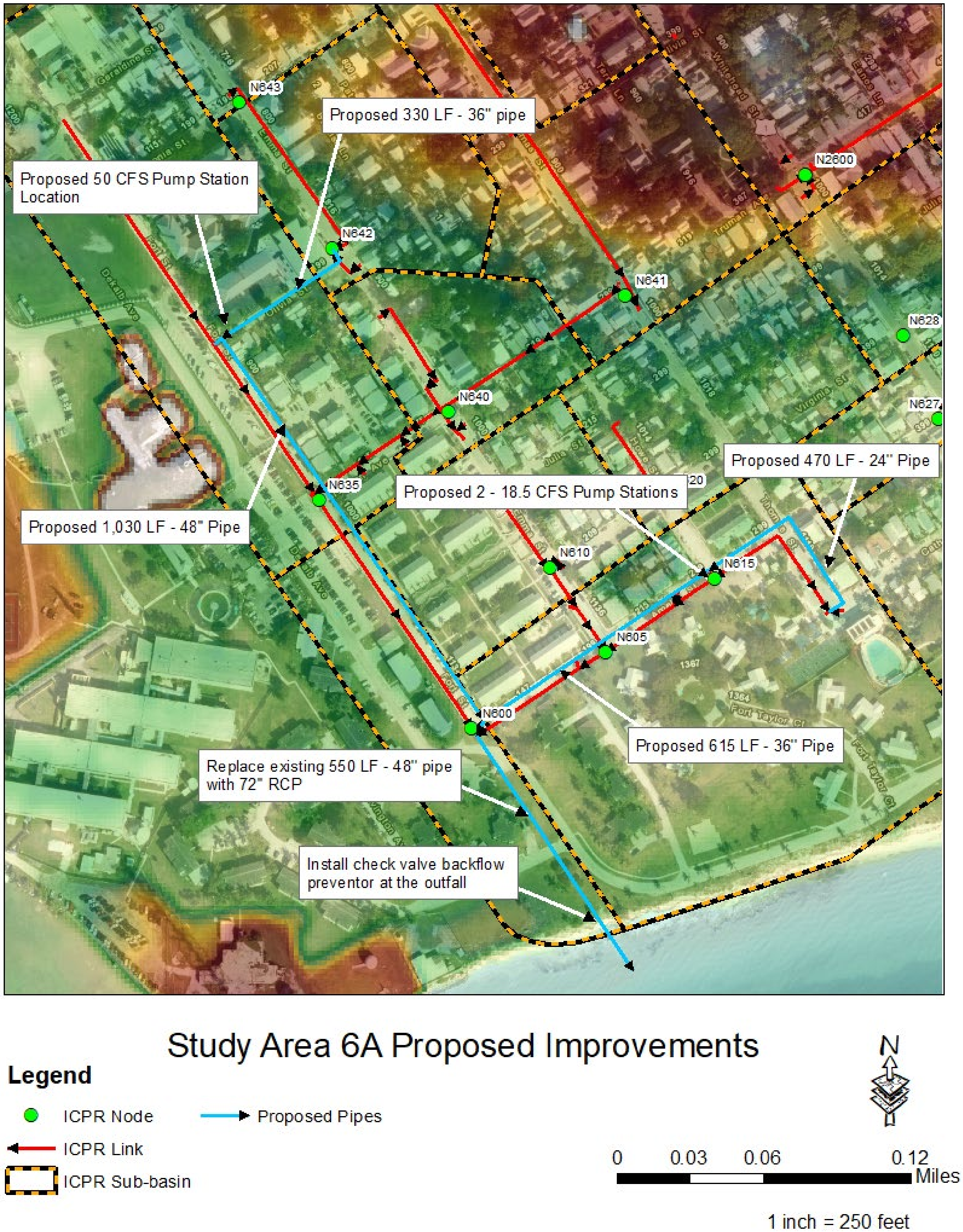


Table 3-28 presents a summary of the simulation results for the 10-year, 24-hour storm with upgrades provided at Study Area 6A.

Table 3-28. Summary of Study Area 6A Simulation Results for the 10-year Storm with Upgraded Conveyance Pipes

Study Area Subbasins	10-Year, 24-hour Elevation (feet NAVD 88)				
	Roadway Low Point (feet)	Existing Conditions, 1-foot Tide	Proposed Solution, 1-foot Tide (no pump)	Existing Conditions, 2.7-foot Tide	Proposed Solution, 2.7-foot Tide (w/pump)
Fort Street at Truman Avenue (N635)	3.01	3.77	2.17	3.96	3.76
Emma Street at Olivia Street (N642)	3.53	3.88	3.16	4.15	3.32
Fort Street at Olivia Street (N600)	2.82	2.77	1.86	3.29	2.84
Emma Street at Amelia Street (N605)	2.02	2.99	2.71	3.34	3.03
Howe Street at Amelia Street (N615)	2.00	2.95	2.60	3.33	2.97

Review of Table 3-28 shows the solution with or without a pump station will provide flood reduction at both target areas, especially at Emma and Olivia Streets. By adding pump stations at two locations in the system, with the 2.7-foot NAVD 88 tidal boundary conditions, the flood conditions will be significantly reduced. Results with the pump for the 2.7-foot tide solidify long-term benefits when SLR continues. The proposed backflow preventer on the outfalls also will reduce negative impacts of SLR.

Class 4 cost opinions were developed for the preferred solution both with and without pumps. Table 3-29 identifies the estimated costs without a pump station in place (costs associated with gravity-based conveyance improvements only), while Table 3-30 identifies the estimated costs with a pump station in place (costs associated with gravity-based conveyance improvements plus pump station).

Table 3-29. Study Area 6A Cost Opinion – without Pump Station

Construction Subtotal (no Pump Station)		$2,648,279
Markups		
Contractors Overhead, General Conditions, Temporary Facilities	15%	$397,242
Contractor Profit	10%	$264,828
Engineering/Design	22%	$582,621
Contingency/Market Volatility	25%	$827,587
Total Including Contingencies		$4,720,557

Table 3-30. Study Area 6A Cost Opinion – with Pump Station

Construction Subtotal (with Pump Station)		$ 11,831,346
Markups		
Contractors Overhead, General Conditions, Temp Facilities	15%	$1,774,702
Contractor Profit	10%	$1,183,135
Engineering/Design	22%	$2,602,896
Contingency/Market Volatility	25%	$3,697,296
Total Including Contingencies		$21,089,374

3.3 Regional/Resiliency Solutions for Flood Relief versus 2.7-foot Tidal Boundary Conditions

In addition to looking at the potential projects in the neighborhoods identified as problem areas, this SWMP also considered a broader review for potential, large-scale, regional strategies that address the future conditions with SLR up to 2.7 feet NAVD 88. The timing of the projections will vary, so the need is not urgent. However, at some point in the future, the ocean levels will reach these high 2.7 feet levels more frequently. This section discusses how bigger infrastructure projects could be formulated to provide resilient solutions for the island.

3.3.1 Master Planned Regional Pump Station Locations

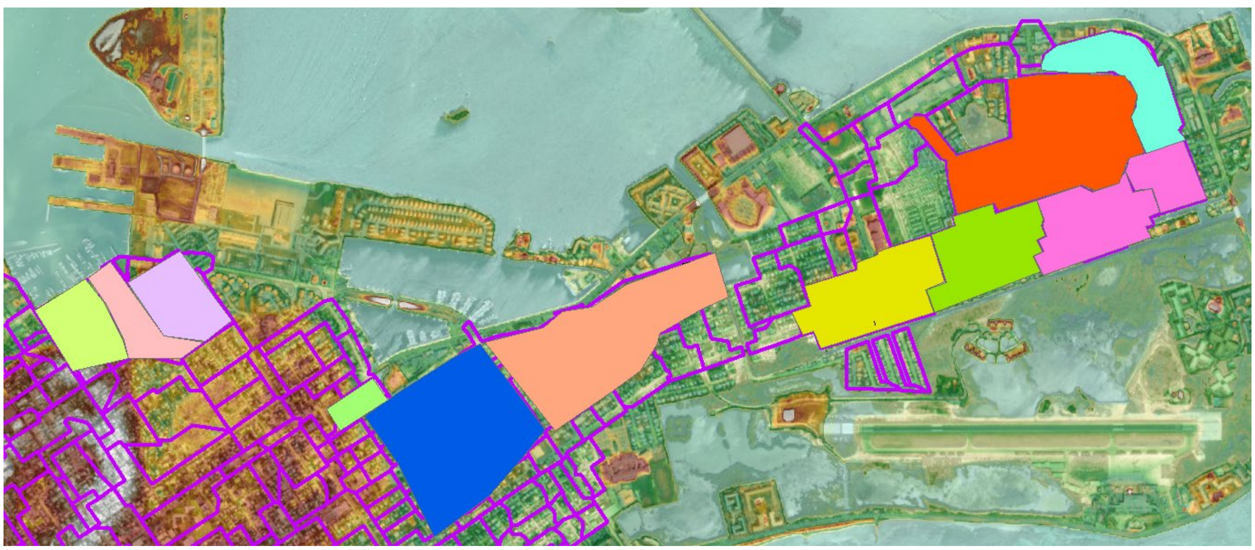
Analysis of the study areas identified that when a future tidal boundary condition of 2.7 feet was inserted into the model, the flood-reduction benefits of isolated regional pump locations were diminished as the neighboring low-lying areas take advantage of the reduced flood stage and quickly drain into the improved area.

From a high-level perspective, regional pump stations may be designed to anticipate peak-flow conditions coming into the storage vault to meet or exceed maximum rainfall intensity anticipated during the determined design storm event for the facility. Because the peak intensity of a 10-year, 24-hour storm event is about 1.7 inches per hour, depending on target LOS and elevations for the area, a 150-cfs peak-flow pump station may be able to handle as much as 85 acres of contributing drainage areas, assuming conveyance through the region is adequately sized (that is, like the individual study areas pipelines discussed in Section 3).

Conceptually, the higher-risk flood areas throughout the City could be delineated into 28- to 90-acre zones, each with its own stormwater vault and regional pump station designed to handle the specific conditions encountered in each basin area. Stormwater storage vaults and pump station locations could be centered near existing gravity outfall locations, so gravity bypass is available in the event that the pump station is not operable. Other overflow locations also should be considered in siting the facilities.

Figure 3-11 provides a high-level, conceptual delineation of several flood-prone areas of the island as divided into potentially suitable regional pump station catchment areas. North Stock Island has a central marsh that could be isolated, but that is more similar to the tidal barriers discussed in Section 3.3.3.

Figure 3-11. Potential Catchment Basins that Could be Served by a Regional Pump Station



Purple lines = Sub-basins; highlighted areas = potential catchment basins that could be served by a Regional Pump Station

Further work would be required to delineate the basins accordingly, select the desired LOS for the master planned stormwater pump stations, and further develop the anticipated stormwater storage vaults (including wet wells) and pump sizes for each given catchment area. Likewise, additional measures should be in place to ensure seawalls and areas adjacent to the coast are high enough (at least 5 feet NAVD 88) to prevent the pumps from pushing against coastal or tidal flooding not related to a storm event. A set of preliminary model simulations comparing a regionally based pump station approach is presented for the proposed catchments that were evaluated as part of this task. These models are conceptual in nature and do not specifically include the individual connections, as proposed and defined in Section 3. Figure 3-12 through Figure 3-14 provide examples of conceptual depictions of the potential benefits from master planning stormwater pump station deployment near three outfalls that service the lower-lying areas of the island. As shown on these graphics, the potential for a regional pump station approach to mitigate rainfall-driven flooding in combination with a 2.7-foot NAVD 88 tidal boundary shows reduction in flood stage and duration and should be further evaluated as the City contemplates long-term future conditions.

Figure 3-12. Regional Pump Station 1 (RP-1) Stage Hydrograph

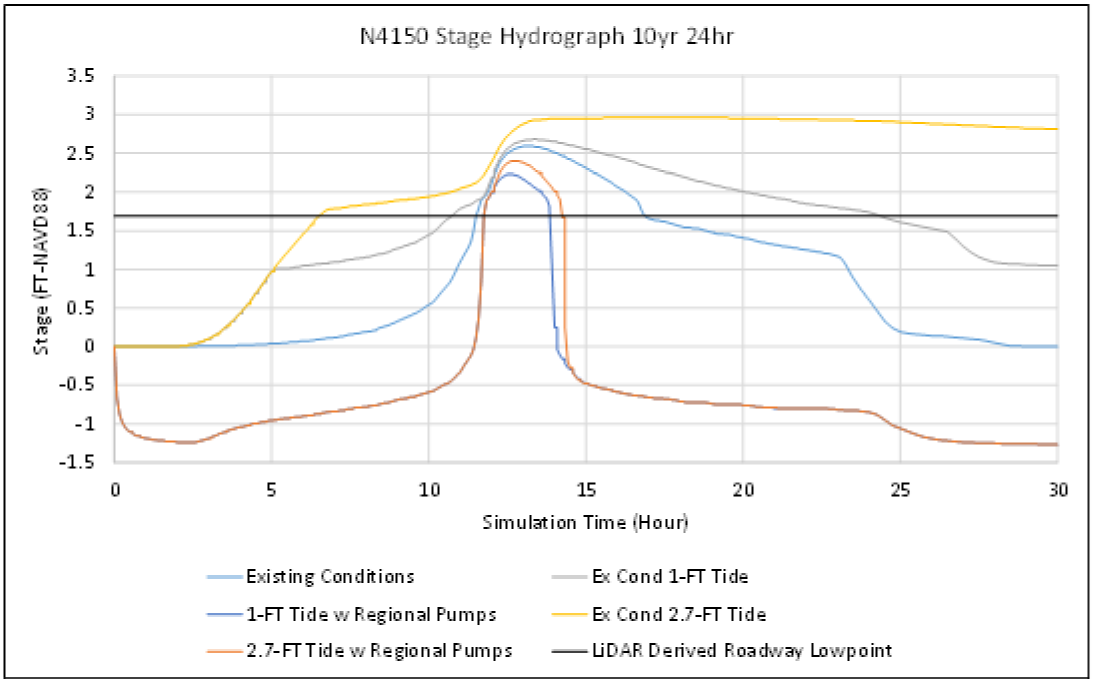


Figure 3-13. Regional Pump Station 2 (RP-2) Stage Hydrograph

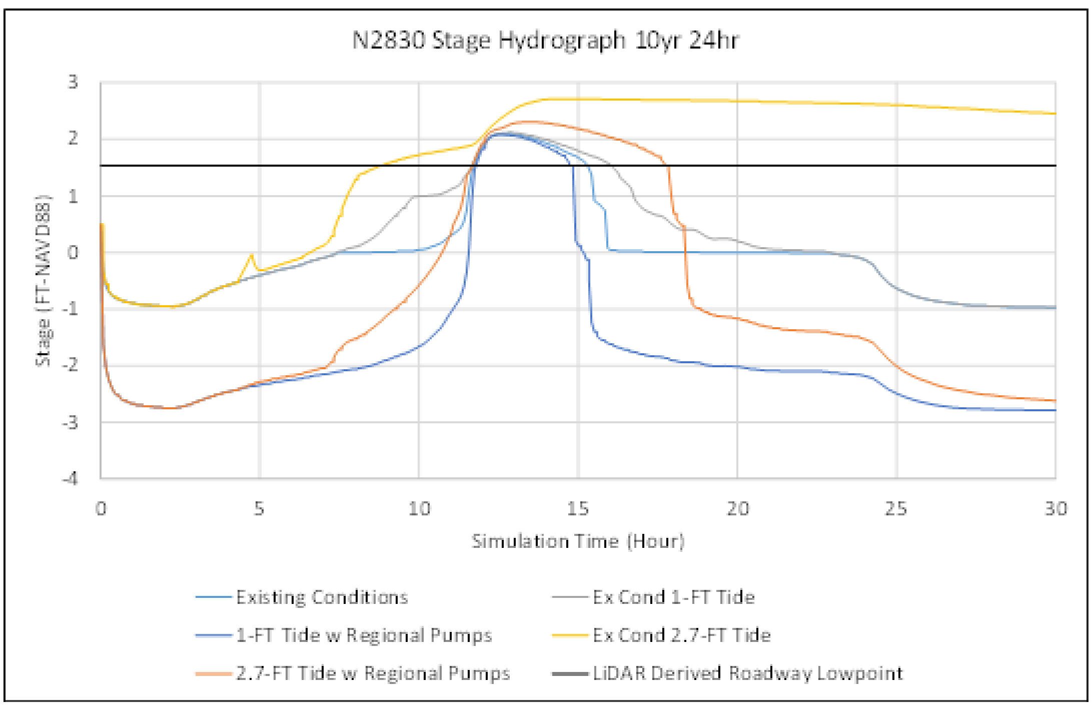
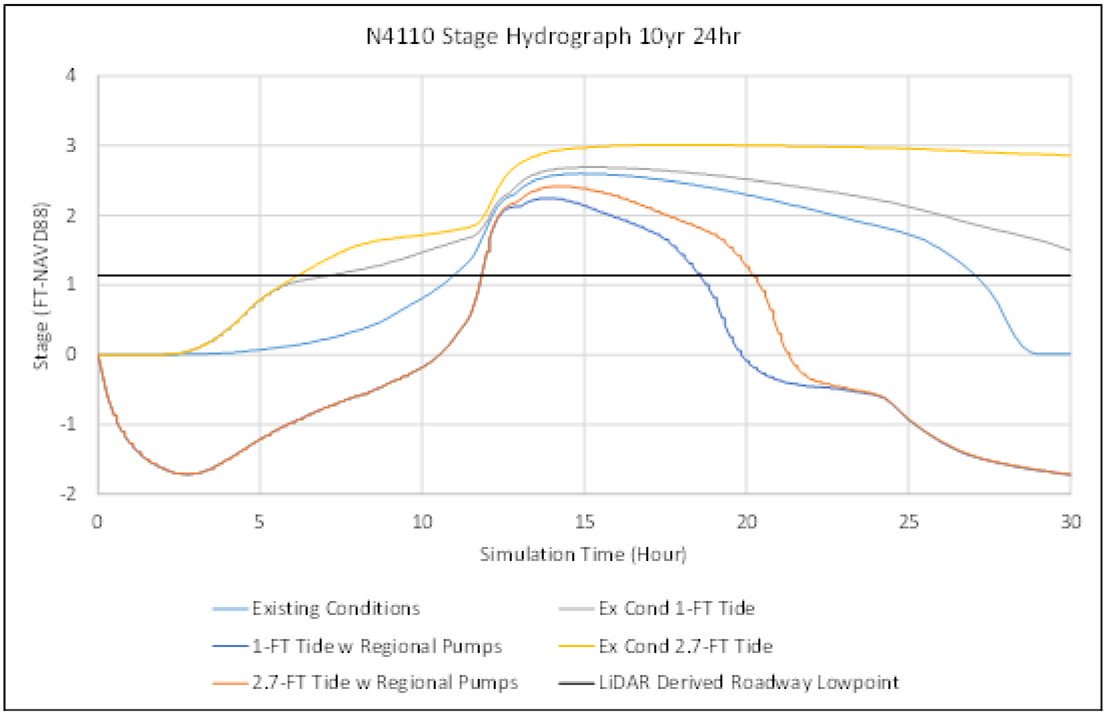


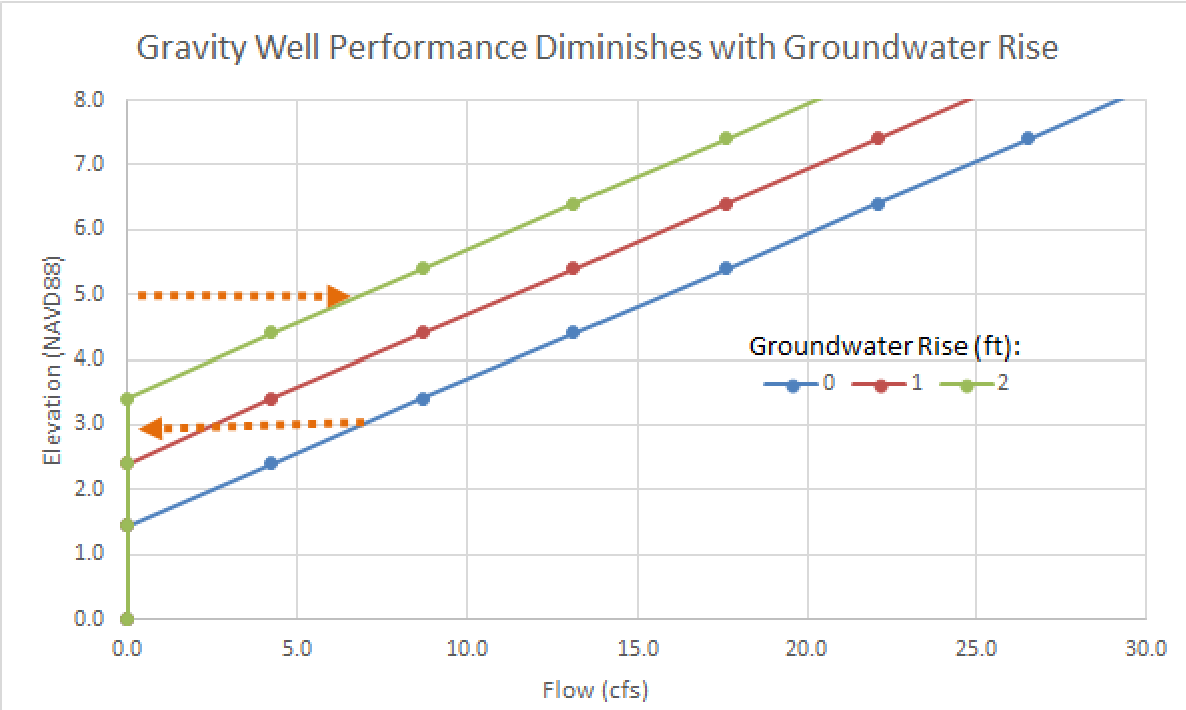
Figure 3-14. Regional Pump Station 3 (RP-3) Stage Hydrograph



3.3.2 Modified Gravity Well Concept

Gravity-fed stormwater injection wells are prevalent throughout the City in low-lying intersections prone to flooding. Figure 3-15 illustrates how higher groundwater may impact the drainage capacity of gravity-fed injection wells in the future. This assumes a linear impact and that transmissivity of the groundwater remains similar. In 2012, the original gravity well rating curves (blue line) led to the recommendation that these types of facilities be located where the ground was at least elevation 2.5 or 3 feet NAVD 88. As groundwater levels have risen, gravity wells are not as effective in pushing freshwater down into the brackish aquifer until the flood stage at the wellhead raises (Figure 3-15). Given the low elevations, greater staging in the future will increasingly put more buildings at risk and make roads unpassable.

Figure 3-15. Typical Gravity Injection Well Capacity in Key West with Different Groundwater Levels



The modified gravity well concept is derived from the idea that many municipal potable water distribution centers use a water tower to generate driving pressure. Water is pumped to a tower that is set at an elevation high enough to provide a reliable gravity-fed source of freshwater to the community. The modified gravity well concept will discharge water to injection well locations in a similar manner, where a tank would be built to provide an aboveground storage area that accepts flow from surrounding stormwater pumps and allows the stormwater to stage up at an elevation high enough to overcome the tide and coincident higher water table elevations. This type of unit would decouple the flood levels on the streets from the rating curve because driving head can be generated. The storage facility could be a custom unit, built to resemble the surrounding buildings and structures, or perhaps commingled with a parking garage or other structure that may be suitable for retrofitting. Stormwater ponds also could be built to hold water at a higher elevation to drain to a gravity well.

The aboveground stormwater storage would be connected to a network of gravity wells to allow stormwater to flow out of the surface water system and reduce flooding, while also providing a water quality benefit. The gravity wells would need to be separated to avoid up-coning.

Although the concept will still involve pumping, it provides an alternate outfall concept that prevents directly pumping to the surrounding waters, given the various water quality initiatives from permitting authorities that have been proven to combat future SLR and tide conditions. Conceptual sketches of these modified gravity wells are included on Figure 3-16 and Figure 3-17.

Figure 3-16. Conceptual Sketch of Modified Gravity Well Concept Structure Location

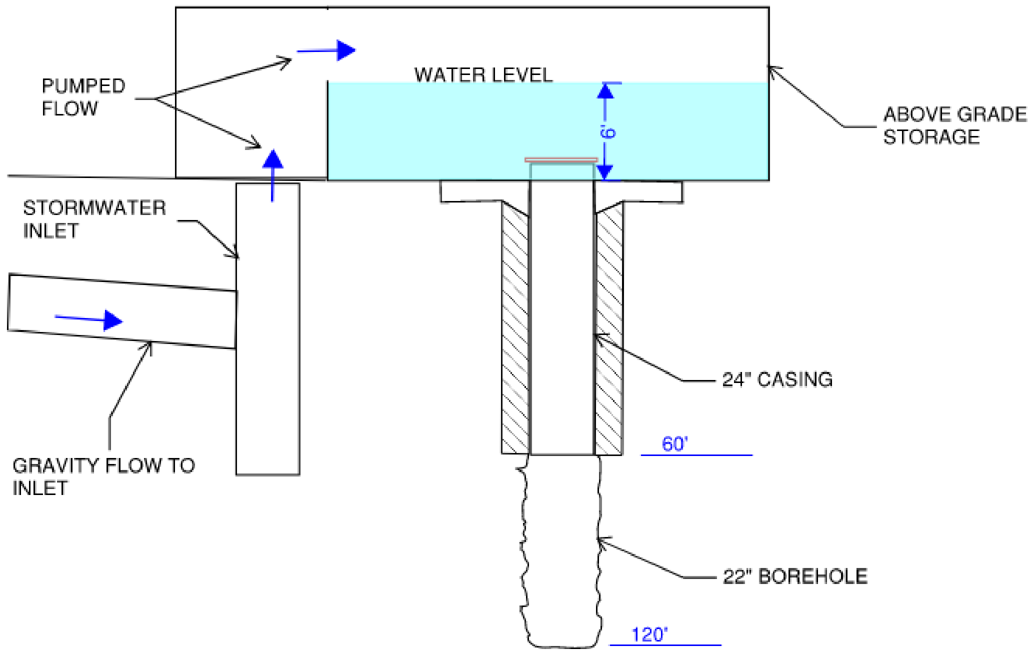
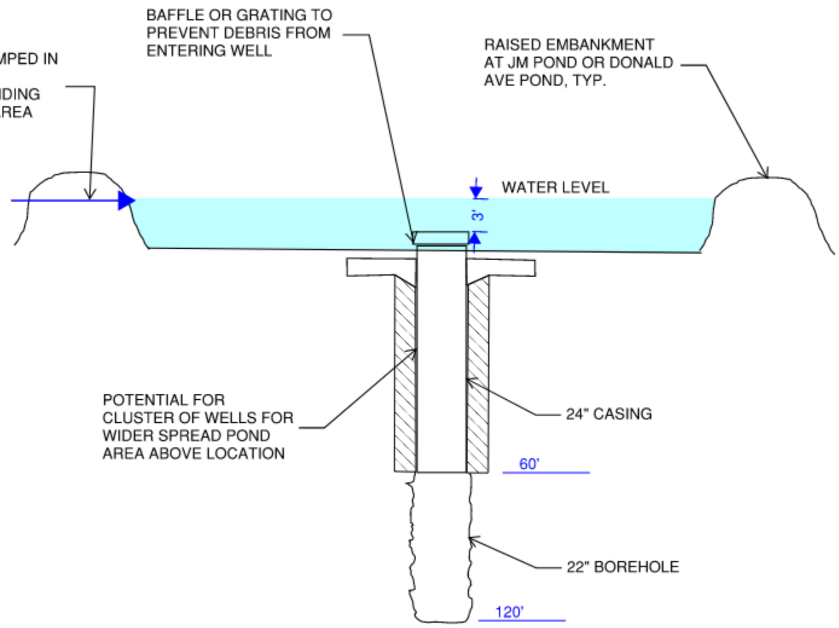


Figure 3-17. Conceptual Sketch of Modified Gravity Well Concept Pond Location



Concept summary description:

- Conceptual storage area would be dependent on pumped flow rates into the facility from the surrounding basins and should be sized to provide enough storage above the target elevation that may avoid surcharging the structure and overly pressurizing the system. For example, if 18 cfs was pumped into the facility, a tank structure equivalent in size to a 3,200-foot building, staged up to 8 to 10 feet high, may provide an equal discharge flow rate to one well location, with as much as 12 minutes of storage to help prevent pressuring the pumped connection to the storage basin. The storage would be subsequently reduced or increased based on the number of gravity wells it connects to and any increase in pumping rates. Baffle walls should be included to obtain some settling for water quality credit. Conceptually, these facilities would be commingled with the regional pump stations to handle the more-frequent, lower-intensity storm events.
- It would be ideal to size the facility large enough to include maintenance and other facilities in a single location or a 32.5- to 65-foot-diameter storage tank. For conceptual modeling purposes, a 1,600-foot facility was anticipated that includes a 90-cfs pumping rate into it and a discharge connection to five gravity wells. This concept would have about 2 minutes of detention storage and require a check valve to bypass flow when required. In general, about 1.5 to 2 minutes of detention is preferred to allow larger sediment to settle.
- Stormwater could be pumped in from several locations (study areas or known flooding locations).
- Outfall from the storage area would be accomplished through multiple gravity well (outfall) connections in a combination of linked pipes, new connections to existing wells, and new wells drilled directly below or within proximity of the newly created storage area.
- Flood-prone areas in higher-elevated regions of the City may not require pumped connections to the storage area. Stormwater inlets and pipes in the system may connect directly to a gravity well with enclosed lid and bypass/equalization pipes routed to coastal outfalls.
- On a smaller scale, roof drains could be retrofitted to connect to enclosed gravity wells in flood-sensitive areas or larger, developed commercial areas.

3.3.3 Tidal Wall or Barrier Wall Concept

This concept is purely to evaluate and consider large-scale, preliminary ideas outside the realm of typical options when dealing with SLR and tidal conditions. The southeastern area of the island's salt marsh is a natural buffer along the south and southeastern extents of the island. This buffer is formed by beachy areas and Highway A1A, and the DEM information appears to show this area is high enough in many areas to meet the 5-foot goal identified in the draft resiliency guidance documentation (Jacobs 2021b). The salt marsh area around the airport that lies between this buffer and areas of the City appears to be mostly isolated from direct coastal influence, except at two bridge locations, one north and another to the east. There is also a culvert to the southeast. Figure 3-18 provides a conceptual view of the subject marshy area described.

Figure 3-18. Salt Marsh and Riviera Canal Area within Southeast Portion of Island

Purple lines = Sub-basins; blue highlighted areas = enhanced stormwater storage areas

Although the salt marsh contains environmentally sensitive areas (mangroves), it also will be adversely affected by SLR and high-tide conditions. The salt marsh is connected to the ocean through the Riviera Canal and through a culvert (near the hotels on the southeast). In this case, water elevations within the blue highlighted storage areas may be controlled by a tide wall or barrier wall at each bridge and culvert location. This also would block recreational boat traffic from homes with docks along Riviera Canal unless this barrier can be along the south edge of the canal (numerous mangroves) or offset somewhat. In some ways, this would turn the salt marsh into a large drainage management storage facility. The drainage system for eastern New Town could go to this new facility. This also would benefit the very vulnerable airport.

This concept would help maintain lower water elevations in the southern area of the island, allowing for controlled outfall capacity in this area, independent of the tidal condition. Large-scale pumps and monitoring could maintain levels. A solution of this magnitude would require a high degree of preliminary study, including a National Environmental Policy Act assessment to assess the feasibility and demonstrate that this would be the least environmentally damaging potential alternative. This may be demonstrated based on goals for regional protection and the future SLR threats.

Similar to the salt marsh on east Key West, North Stock Island also has a large marshy lagoon that is already partially isolated through a series of culverts. It too has extensive mangroves; however, this area could be blocked and have a pump station to keep the lagoon levels at lower elevations to facilitate drainage.

3.3.4 Oversized Long Box Culvert Runs

Similar to a canal providing stormwater outfall capacity to inland areas, large box culvert runs can interconnect into the low-lying areas of the City with specific measures built in to prevent backflow conditions from the tide into the system. The City has recently installed an 81-inch-diameter arch pipe west of Donald Avenue, enclosing an open channel system and achieving a similar objective in the area. This pipe was installed to close in a segment of canal that could not be maintained and presented a safety issue. However, oversized pipes are another way to provide additional storage.

These box culverts could be designed to run through lower-traffic areas of the City and be used as a stormwater storage vault connected to a regional pump station at the outfall end of the culvert. This solution was conceptually modeled to show Study Areas 4A and 4B meet or exceed the 10-year, 24-hour LOS criteria. However, when modeled with a 2.7-foot tidal boundary condition, the improvements are reduced.

4 Summary of Projects

The analyses conducted during the Phase 2 Stormwater Master Plan were tasked to find solutions for the long term, considering SLR. The evaluations described in Section 3 have identified several potential solutions that are unique to each study area evaluated. With SLR, large portions of the study areas are vulnerable and pump stations will be required. To be effective, large pipe networks will have to feed the new pump stations. In summary, future solutions are going to require big projects. Given the total potential scale, the City may wish to combine sub-basin areas and consider a more regional approach with pump stations. Additional big picture strategies were identified in Section 3.3.

From the modeling and subsequent analyses in the six primary study areas (Section 3.2), the following recommendations are being made for the City's consideration in its stormwater management for near term- and long-term future conditions. As SLR continues to impact the City (that is, 1 foot or more of tidal elevation), solutions will need to be put in place to reduce flood elevations and durations. The solutions presented within this report have shown that the City can expect both reduced flood elevations and durations from the proposed improvements at each location. The ultimate timing and phasing of the solutions could be further expanded upon in future master plans, but the current recommendation is that the bulk of the proposed improvements be implemented sooner rather than later.

In general, the individual study areas simulated in Phase 2 exhibit the following similar characteristic, whereby the proposed solution (individually described in Section 3.2) is a combination of increased pipe sizes, as well as rerouting existing stormwater conveyance to an outfall with large capacity (such as 150 cfs). All study areas were evaluated with and without pumping. The results are favorable to reduce flooding in portions of the study areas without pumps under current 1.0-foot tide elevations. However, against a 2.7-foot NAVD 88 tide, pumps are necessary to reduce roadway and structure flooding.

Study Area 1A Summary: 2nd Street to 5th Street between Patterson Avenue and Harris Avenue

- The proposed solution is a combination of increased pipe sizes, as well as rerouting existing stormwater conveyance to a larger outfall. This study area was evaluated with and without pumping. The results are favorable to reduce flooding in portions of the study area without pumps. As the existing flooding is within 0.6 foot of the roadway low points and this is a local road, road raising has not been considered as a part of the preferred solution costs for this alternative.

Study Area 1B Summary: Patterson Avenue between 5th Street and 7th Street

- Because of the low elevations along the roadway and the size of the total contributing drainage area surrounding this sub-basin, conveyance pipe and stormwater pump station sizing in this area is directly influenced by both the desired flood LOS for the area (whether any minor flooding is allowed during the subject storm event) and any ability to raise roadway elevations through the area. The DEM topography indicates that the low elevations continue through some of the residential parcels, creating a challenge for either option.

Study Areas 2A, 2B, and 2C Summary: Tropical Street to George Street Neighborhood Drainage

- As there are recent stormwater projects constructed in this area, the preferred alternative includes leaving those improvements in place and raising road elevations at the low-lying intersection to a minimum 1.5 feet NAVD 88. This will prevent large-scale removal and replacement of recent construction work and limit flood depth and duration during the 10-year, 24-hour storm event. Still, more conveyance and a pump station are recommended. A larger pump station with storage could be built at Bayview Park and consolidate other inlets in the vicinity, with discharge to Jose Marti Pond. The intersection of United Street and Ashby Street sees an incidental benefit from the proposed improvements to Study Areas 2A and 2B to the west.

Study Area 3A Summary: West Riviera Drive Improvements

- The preferred solution makes use of the existing larger outfalls from Flagler Avenue and services the area with its own expanded outfalls that handle both Riviera Drive and overflow from flooding on Flagler Avenue. When paired with the road raising included with this solution, flooding during the 10-year, 24-hour event will be limited to within 6 inches for a limited duration when subjected to the 1.0-foot NAVD 88 tide boundary condition. Some road raising also will reduce flood depth and duration for the low areas along Riviera Drive currently serviced by outfalls that are not easily accessible as they pass through private property leading to the canal. Directly connecting these areas to the new outfalls does not appear to provide a suitable reduction in flood stage and duration by comparison.

Study Areas 4A and 4B Summary: Eagle Avenue at 20th Street and 18th Terrace at Donald Avenue east to 20th Terrace

- Several alternatives were evaluated for this area that focus on routing the conveyance through shared outfalls or oversizing the conveyance system to either eliminate roadway flooding for the 10-year, 24-hour event or reduce the peak stage to limit the anticipated inundation to less than 0.25 foot above the lower roadway segments. These alternatives proved that, although possible, there is a diminishing return on value in oversizing the pipes to the level required. Likewise, the area will continue to be subjected to SLR that will further degrade the expected peak flood stage reduction without the use of a pump station. Because of the flatness of this area, a pump station would be recommended sooner than later.

Study Area 4C Summary: Glynn Archer Jr. Drive between Glynn Archer Jr. Street and Duck Avenue

- Simulations indicate sub-basins along Glynn R. Archer Jr. Drive are unable to be fully removed from flooding during the 10-year, 24-hour storm event with the proposed improvements in place when subject to a tidal boundary condition of 2.7 feet NAVD 88. To fully remove the subject areas from the 10-year, 24-hour flood event, the proposed improvements include raising the road elevations where possible, as the roadway locations below the flood stage are located in isolated areas where the opportunity may be available. A review of the elevation information included in the DEM identifies that a minimum road elevation of 1.5 feet NAVD 88 may be suitable to prevent flooding in the localized areas by more than 6 inches when subject to the 10-year, 24-hour storm event.

  It is important to note that the west outfall location presents an additional challenge when compared to the tidal boundary condition of 2.7 feet NAVD 88. The existing roads in the vicinity of the outfall are currently lower than the future tide boundary condition. These areas should be addressed independently as a part of meeting the resiliency standards outlined in the draft resiliency policy (Jacobs 2021b). An alternative solution that directs flow to the north through a new outfall proposed through Kennedy Drive at North Roosevelt Boulevard intersection was evaluated with Study Area 4D. The additional pump and outfall proposed with Study Area 4D provide a mutual benefit to the area that may prevent the Glynn R. Archer Jr. Drive area from flooding. Because that solution includes a new outfall location, it may only be considered upon the desire to move forward with the solution proposed in Study Area 4D.

Study Area 4D Summary: Northside Drive to Duck Avenue between 15th Terrace and 17th Terrace

- Simulations show that constructing a large new outfall that services the Donald Avenue area will provide a benefit to the lower-lying areas along the eastern part of the City. When in place, other study area solution alternatives may be modified to take advantage of the additional stormwater conveyance capacity through this area. For example, Donald Avenue at 19th Street and Donald Avenue at 16th Terrace could both be modified with larger conveyance pipes into the Donald Avenue area to remove their subject basins from the 10-year, 24-hour flood stage.

Study Area 5A Summary: Frances Street to White Street between Eaton Street and Fleming Street

- A portion of this study area contains commercial buildings, which negated additional evaluation of potential existing pipe size increase for the particular location (near White and Grinnell Streets). However, the preferred gravity outfall route for the stormwater system is to tie directly to the 4-foot by 6-foot box culvert located north of the intersection on White Street. A proposed pump station may be located anywhere from the White Street at Eaton Street intersection to the outfall corridor down Grinnell Street or adjacent to it. The pump station will require an outfall with which to connect. There is currently a vault near the ferry that is near the terminus of the culvert. The pipe connections along Mustin Street to the Grinnell outfalls are not well understood. There may be a better way to connect this box culvert to an outfall. The connections near the ferry could be further investigated with closed-circuit television to better assess the connectivity. Using the culvert or an increased pump vault capacity may help reduce stormwater pump station size requirements to provide a benefit for long-term future tide boundary conditions. Alternatively, a pump station and new outfall down White Street may be pursued, but this will cross federal property. The project estimated discharge piping from the pump station along the longer path to the outfalls for planning purposes.

Study Area 6A Summary: Southern Bahama Village

- The existing outfall along Fort Street could be better used in the near term with better connections from the side streets to the east. However, the future SLR will make a pump station for the neighborhood necessary. The landscape near Olivia Street is relatively high when compared to the southern end near Catherine Street. For this area, two smaller pump stations were recommended. A force main from the north can be added along Fort Street and then the smaller pump station near the south can be tied into it.

Additional analyses were conducted for regional and resiliency-based solutions (Section 3.3). The culmination of the analyses is summarized in the following paragraph, with simple benefit-cost analysis based on the assumption that reductions of peak stage for the 10-year, 24-hour event will provide an LOS for roadway travel and prevention of structural damage.

- From the simulations and analyses presented herein, regional pump station solutions should provide reasonable reductions in flood stages and dramatic decreases in flood duration. Regional pump stations, in conjunction with upsized conveyance features may provide large-scale (citywide) benefits that reduce both peak stage and duration of flooding. This regional-scale pump station simulation will need further review with simulations that combine some or all of the preferred solutions proposed in Section 3 to understand the full complement of flood reduction these will provide against a long-term future condition tide of 2.7 feet.

Other regional/resiliency-oriented concepts were developed but have not been explicitly simulated during this SWMP. Depending on the specific location, they may provide additional benefits or complement those provided herein. These ideas were included to provide the City with additional considerations as the island faces SLR that may affect large portions of the City.

- A modified gravity well with an aboveground holding tank could achieve additional water quality benefits as well as a cost benefit by using smaller pump configurations as opposed to regional or single/point pressure (injection) well locations.
- A tide barrier, like a combination of sea walls and higher roads, at select locations would prevent increasing tides associated with SLR from moving inland. Preventing high tides from inland migration should have widespread positive benefits as related to stormwater management. Drainage from behind the barrier would still need to be pumped to the ocean.
- Oversized long box culvert runs would provide additional conveyance of stormwater throughout the City system, which in turn will provide more stormwater storage. pump stations would need to be located to accommodate the flood reductions necessary against increased tidal elevations.

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Appendix A
Existing Condition Simulation Results

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
DuncAsh	NA	Ashby Street	Duncan Street	1.65	2.11	2.33	2.64
N100	B100	Between Thompson and Leon Streets	South of Atlantic Blvd.	1.82	1.95	2.08	2.35
N1000	NA	7th Street	North of Patterson Avenue	0.65	0.72	0.83	0.95
N1005	B1005	7th Street	Patterson Avenue	1.91	2.22	2.63	2.71
N1010	NA	6th Street	North of Patterson Avenue	0.74	0.80	0.91	1.04
N1015	B1015	6th Street	Patterson Avenue	1.90	2.01	2.12	2.34
N1020	B1020	6th Street	Fogarty Avenue	1.93	2.03	2.13	2.36
N1025	B1025	6th Street	Harris Avenue	2.63	2.68	2.73	2.79
N1030	B1030	7th Street	Fogarty Avenue	1.96	2.04	2.14	2.36
N110	NA	PS – Patricia and Ashby	Patricia Street	1.53	1.63	1.74	1.94
N110002	B130002	Josephine Street	Venetia Street	1.80	1.92	2.04	2.30
N110003	B130003	Leon Street	Laird Street	2.43	2.49	2.56	2.67
N110004	B130004	Thompson Street	Laird Street	2.08	2.13	2.20	2.38
N110005	B130005	Josephine Street	Atlantic Blvd.	1.79	1.92	2.04	2.30
N110006	B130006	Bertha Street	Venetia Street	1.75	1.81	1.94	2.27
N110007	B130007	George Street	Patricia Street	1.54	1.64	1.74	1.94
N110008	B130008	Thompson Street	Rose Street	1.84	1.97	2.10	2.36

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N110009	B130009	George Street	Blanch Street	1.80	1.93	2.05	2.32
N110010	B130010	Ashby Street	Rose Street	1.82	1.96	2.09	2.35
N110011	B130011	George Street	Venetia Street	1.81	1.93	2.05	2.32
N110012	B130012	Ashby Street	Johnson Street	1.88	2.00	2.11	2.35
N110013	B130013	Ashby Street	Atlantic Blvd.	1.82	1.95	2.08	2.34
N110a	NA	Ashby Street	Patricia Street	1.80	1.94	2.08	2.34
N110b	NA	Ashby Street	Patricia Street	1.80	1.94	2.08	2.34
N110c	B130	Ashby Street	Patricia Street	1.80	1.94	2.08	2.34
N110i	NA	Ashby Street	Patricia Street	1.80	1.94	2.08	2.34
N1120	B1120	Duval Street	Eaton Street	4.00	4.54	5.13	6.14
N1120b	B1120b	Duval Street	Fleming Street	4.00	4.54	5.12	6.22
N1120c	B1120c	Simonton Street	North of Fleming Street	8.22	8.34	8.42	8.50
N1130	B1130	Duval Street	Fleming Street	5.19	6.19	7.25	7.80
N1140	B1140	Duval Street	Angela Street	6.27	6.72	7.22	7.77
N1150	B1150	Whitehead Street	Angela Street	4.73	5.39	5.57	5.97
N1160	B1160	Simonton Street	Angela Street	5.53	6.43	6.84	7.44
N1160b	B1160b	Duval Street	Angela Street	6.81	7.17	7.47	8.05
N1170	B1170	Duval Street	Petronia Street	8.44	8.69	8.80	8.94
N1180	B1180	Duval Street	Southard Street	5.38	6.41	7.45	7.92

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N1190	B1190	Whitehead Street	Caroline Street	2.82	2.88	2.94	3.04
N200	B200	White Street	Between Atlantic Blvd. and Casa Marina Court	0.65	0.66	0.66	2.25
N2000	B2000	Whitehead Street	Front Street	2.40	2.52	2.63	2.87
N2010	B2010	Whitehead Street	Greene Street	2.34	2.51	2.61	2.86
N2020	B2020	Whitehead Street	North or Caroline Street	2.18	2.50	2.65	2.86
N210	B210	White Street	Laird Street	2.43	2.54	2.64	2.89
N2100	B2100	Duval Street	Front Street	0.58	1.06	1.34	2.02
N2100b	B2100b	Wolfson Street	Front Street	1.54	1.64	1.72	1.91
N2100c	B2100c	Fitzpatrick Street	Front Street	2.40	2.52	2.63	2.87
N2110	B2110	Simonton Street	PS - Simonton	0.93	1.15	1.34	2.02
N2120	B2120	Duval Street	Between Greene and Front Streets	2.42	2.55	2.66	2.91
N2120b	B2120b	Ann Street	Greene Street	2.08	2.14	2.20	2.29
N2130	B2130	Duval Street	Caroline Street	2.76	2.90	3.02	3.23
N2130b	B2130b	Ann Street	Caroline Street	2.34	2.52	2.76	3.09
N2135	B2135	Duval Street	Between Caroline and Eaton Streets	3.33	3.56	3.72	3.95
N2140	B2140	Simonton Street	Caroline Street	2.14	2.32	2.52	3.42
N2140b	B2140b	Simonton Street	North of Eaton Street	5.43	5.87	6.56	7.50

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N215	B215	Whalton Street	Johnson Street	2.95	3.18	3.29	3.48
N220	B220	Florida Street	Laird Street	2.39	2.52	2.63	2.88
N2200	B2200	Elizabeth Street	Greene Street	0.27	0.41	0.61	0.99
N2200b	B2200b	Elizabeth Street	North or Caroline Street	1.90	1.95	2.03	2.20
N230	B230	White Street	Von Phister Street	3.64	3.68	3.71	3.78
N2300	B2300	William Street	Caroline Street	1.62	1.72	1.83	2.09
N2300b	B2300b	Elizabeth Street	South of Caroline Street	2.12	2.24	2.32	2.45
N2300c	B2300c	West of William Street	North of Fleming Street	2.88	3.26	3.71	4.81
N2301	NA	East Elizabeth	Caroline Street	1.73	1.81	1.91	2.13
N2302	NA	Peacon Lane	Caroline Street	1.65	1.74	1.85	2.10
N2310	B2310	William Street	North or Caroline Street	1.16	1.43	1.64	2.04
N235	B235	White Street	Von Phister Street	4.15	4.21	4.24	4.31
N240	B240	Whalton Street	Von Phister Street	3.33	3.50	3.65	3.94
N2400	B2400	Margaret Street	Caroline Street	1.60	1.96	2.19	2.33
N2400b	B2400b	West of Margaret Street	North of Fleming Street	2.47	2.49	2.52	2.59
N2400c	B2400c	Roberts Lane	Sawyers Lane	2.14	2.53	2.66	2.77
N2410	B2410	Margaret Street	North of Caroline Street	1.31	1.69	1.90	2.15
N245	B245	Grinnell Street	Von Phister Street	3.33	3.50	3.65	3.94
N250	B250	Grinnell Street	Johnson Street	2.75	2.88	3.03	3.29

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N2500	B2500	Grinnell Street	Boundary, North of Trumbo Road	0.77	1.14	1.48	2.18
N2500D	B2500D	Frances Street	James Street	0.84	1.18	1.41	1.74
N2501	B2501	Grinnell Street	Trumbo Road	0.46	0.66	0.83	1.20
N250F	B250F	Grinnell Street	Trumbo Road	1.84	1.96	2.10	2.33
N250SP	B250SP	Trumbo Road	James Street	2.75	2.92	3.07	3.46
N250SP_1	N/A	Ferry Term Parking	N/A	0.03	0.05	0.07	0.12
N2510	B2510	White Street	Eaton Street	2.06	2.19	2.31	2.59
N2515	B2515	White Street	North of Eaton Street	0.04	0.07	0.09	0.17
N2516	B2516	Parking lot between White and Frances Streets	North of Eaton Street	2.07	2.19	2.31	2.60
N2520	B2520	Frances Street	Eaton Street	2.02	2.15	2.26	2.48
N2530	B2530	Grinnell Street	Eaton Street	2.11	2.20	2.29	2.48
N2540	B2540	Grinnell Street	Fleming Street	3.86	4.17	4.34	4.47
N2550	B2550	Southard Street	Margaret Street	4.43	4.51	4.59	5.04
N2555	B2555	William Street	Fleming Street	3.94	4.66	5.58	7.63
N2560	B2560	Margaret Street	Angela Street	4.19	4.45	4.69	5.09
N2563	B2563	Passover Lane	Windsor Lane	4.42	4.61	4.81	5.15
N2567	B2567	William Street	Windsor Lane	4.82	5.01	5.13	5.31

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N2570	B2570	Passover Lane	Olivia Street	5.15	5.23	5.34	5.55
N2600	B2600	Whitehead Street	Truman Avenue	7.44	7.52	7.57	7.64
N2610	B2610	Center Street	Truman Avenue	6.26	6.36	6.47	6.69
N2700	B2700	Between Florida and Pearl	Truman Avenue	1.83	1.94	2.06	2.34
N2705	B2705	Jose Marti Drive/Eisenhower Drive	Truman Avenue	1.83	1.88	1.94	2.21
N2710	B2710	Georgia Street	Truman Avenue	3.02	3.31	3.71	4.66
N2730	B2730	Varela Street	Truman Avenue	2.91	3.07	3.20	3.42
N2740	B2740	Grinnell Street	Truman Avenue	3.45	3.95	4.15	4.50
N2750	B2750	Passover Lane	Truman Avenue	4.25	5.19	5.38	5.67
N2800	B2800	Pearl Street	Between Eliza and Virginia Streets	2.04	2.13	2.19	2.48
N2802	B2802	Jose Marti Drive	Between Virginia and Truman Avenue	1.38	1.57	1.78	2.17
N2807	NA	Pearl Street	Eliza Street	1.43	1.57	1.79	2.17
N2810	B2810	Leon Street	Catherine Street	1.00	1.45	1.92	2.48
N2820	B2820	Thompson Street	Catherine Street	1.92	2.16	2.35	2.65
N2830	B2830	Thompson Street	Seminary Street	2.30	2.47	2.63	2.91
N2830b	NA	Thompson Street	United Street	2.10	2.25	2.42	2.70
N2832	B2832	Thompson Street	Washington Street	2.70	2.90	3.00	3.19

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N2834	B2834	Thompson Street	Von Phister Street	2.31	2.54	2.83	3.33
N2836	B2836	Leon Street	Von Phister Street	2.72	2.96	3.12	3.37
N2838	B2838	Tropical Street	Von Phister Street	2.77	2.96	3.11	3.37
N2840	B2840	Leon Street	South Street	2.90	3.04	3.16	3.38
N2842	B2842	Tropical Street	Washington Street	2.97	3.05	3.11	3.37
N2844	B2844	Tropical Street	South Street	2.27	2.49	2.77	3.22
N2846	B2846	Tropical Street	Seminary Street	3.31	3.39	3.46	3.56
N2847	B2847	Pearl Street	Catherine Street	2.66	2.71	2.75	2.80
N2850	B2850	Florida Street	Catherine Street	2.62	2.97	3.40	4.02
N2852	B2852	Pearl Street	United Street	2.09	2.27	2.49	2.93
N2855	B2855	Florida Street	United Street	3.11	3.56	3.94	4.07
N2860	B2860	Georgia Street	Catherine Street	2.63	2.97	3.40	4.10
N2865	B2865	Georgia Street	United Street	2.83	3.20	3.68	4.62
N2870	B2870	White Street	Catherine Street	3.30	3.83	4.49	5.32
N2880	B2880	Varela Street	Catherine Street	3.33	3.89	4.62	5.68
N2883	B2883	Packer Street	Catherine Street	2.68	3.02	3.42	4.39
N2887	B2887	Grinnell Street	Virginia Street	4.44	5.31	5.57	5.75
N2890	B2890	Margaret Street	Catherine Street	3.40	3.96	4.73	5.44
N2892	B2892	Royal Street	Catherine Street	2.69	3.03	3.60	5.60

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N2895	B2895	William Street	Catherine Street	4.79	5.76	6.34	6.46
N2900	B2900	Eisenhower Drive	Newton Street	1.97	2.03	2.09	2.20
N300	B300	Reynolds Street	Atlantic Blvd.	0.79	1.06	1.27	1.50
N3000	B3000	George Street	Catherine Street	0.72	1.61	2.00	2.38
N3010	B3010	George Street	United Street	0.97	1.86	2.26	2.84
N3020	B3020	Ashby Street, near PS-George Street	Catherine Street	1.36	2.03	2.27	2.60
N3030	B3030	Ashby Street	United Street	2.03	2.20	2.39	2.68
N3040	B3040	Ashby Street	Seminary Street	2.10	2.25	2.42	2.70
N3050	B3050	George Street	South Street	2.48	2.66	2.76	2.92
N3060	B3060	Ashby Street	Washington Street	1.53	1.58	1.68	1.94
N310	B310	Reynolds Street	Von Phister Street	3.44	3.59	3.72	3.96
N3100	B3100	1st Street	North Roosevelt Blvd.	0.70	0.77	0.85	0.97
N3110	B3110	1st Street	Roosevelt Drive	1.50	1.67	1.86	2.15
N3115	B3115	1st Street	Patterson Avenue	1.83	1.97	2.10	2.31
N3120	B3120	1st Street	Seidenberg Avenue	1.85	2.00	2.16	2.55
N320	B320	Reynolds Street	South Street	4.86	4.93	5.00	5.13
N3200	B3200	4th Street	Patterson Avenue	1.82	1.94	2.06	2.28
N3210	B3210	3rd Street	Patterson Avenue	1.85	1.98	2.10	2.31

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N3220	B3220	4th Street	Fogarty Avenue	1.97	2.06	2.17	2.36
N3225	NA	3rd Street	Fogarty Avenue	1.88	2.00	2.12	2.32
N3230	B3230	5th Street	Fogarty Avenue	1.97	2.07	2.17	2.37
N3235	B3235	5th Street	Seidenberg Avenue	2.70	2.73	2.76	2.84
N3240	B3240	4th Street	Harris Avenue	2.03	2.11	2.19	2.38
N3250	B3250	3rd Street	Harris Avenue	1.88	2.00	2.12	2.32
N3260	B3260	2nd Street	Fogarty Avenue	1.89	2.00	2.12	2.33
N3300	B3300	Between 5th and 6th Streets	Flagler Avenue	1.31	1.49	1.63	1.90
N3310	B3310	4th Street	Flagler Avenue	1.92	2.06	2.16	2.35
N3320	B3320	3rd Street	Flagler Avenue	2.23	2.33	2.42	2.60
N3330	B3330	2nd Street	Flagler Avenue	2.53	2.61	2.68	2.86
N3340	B3340	Dennis Street	Venetia Street	0.90	1.12	1.28	1.56
N3345	B3345	1st Street	Flagler Avenue	2.81	2.93	3.02	3.20
N3350	B3350	George Street	Flagler Avenue	3.00	3.15	3.26	3.45
N3360	B3360	Thompson Street	Flagler Avenue	3.10	3.25	3.38	3.62
N3370	B3370	Between Leon and Tropical Streets	Flagler Avenue	3.13	3.28	3.41	3.66
N3375	NA	Between Tropical and White Streets	Flagler Avenue	3.13	3.28	3.41	3.66

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N3400	B3400	8th Street	Flagler Avenue	1.77	1.86	1.94	2.10
N3410	B3410	7th Street	Flagler Avenue	1.97	2.04	2.11	2.26
N3500	B3500	West of the 9th Street Canal	Patterson Avenue	1.43	1.48	1.52	1.61
N3600	B3600	11th Street	Flagler Avenue to Riviera Drive	1.47	1.73	1.78	2.02
N3605	NA	11th Street	Flagler Avenue	1.52	1.77	1.84	2.07
N3610	NA	Between 10th and 11th Streets	Flagler Avenue	1.65	1.89	2.01	2.22
N3615	B3610	Between 10th and 11th Streets	Flagler Avenue	2.27	2.49	2.74	2.93
N3620	B3620	12th Street	Flagler Avenue	2.42	2.53	2.60	2.69
N3700	B3700	11th Street	North of Patterson Avenue	0.96	1.29	1.60	1.92
N3710	B3710	12th Street	North of Patterson Avenue	1.28	1.69	2.06	2.33
N3720	B3720	12th Street	Fogarty Avenue	2.00	2.15	2.28	2.46
N3730	B3730	11th Street	Fogarty Avenue	2.14	2.21	2.27	2.40
N3740	B3740	13th Street	About Patterson Avenue	1.42	1.85	2.24	2.54
N3750	B3750	13th Street	North of Patterson Avenue	1.81	2.10	2.34	2.58
N3760	B3760	13th Street	Northside Drive	2.25	2.42	2.58	2.84

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N3765	NA	Between 14th Street and Kennedy Drive	Northside Drive	2.31	2.47	2.62	2.90
N3770	B3770	14th Street	Northside Drive	2.31	2.47	2.62	2.90
N3780	B3780	14th Street	Nr. Stadium MH Park	2.05	2.20	2.37	2.78
N3790	B3790	14th Street	Nr. Stadium Apts.	2.34	2.48	2.63	2.90
N3800	B3800	Rivera Street (15th)	Flagler Avenue to Riviera Drive	1.56	1.75	1.93	2.28
N3810	B3810	16th Street	Flagler Avenue	1.95	2.14	2.28	2.53
N3820	B3820	14th Street	Flagler Avenue	2.22	2.44	2.54	2.75
N3830	NA	West of 14th Street	Flagler Avenue	2.28	2.47	2.57	2.77
N3835	B3830	Between 13th and 14th Streets	Flagler Avenue	2.33	2.48	2.58	2.78
N3837	B3837	13th Street	Riviera Drive	2.35	2.48	2.58	2.79
N3900	B3900	18th Street	Flagler Avenue	1.88	2.07	2.24	2.56
N3902	NA	West of 18th Street	Flagler Avenue	1.89	2.08	2.25	2.57
N3910	B3910	17th Street	Flagler Avenue	2.25	2.34	2.42	2.59
N3912	B3912	18th Street	Riviera Drive	2.19	2.34	2.42	2.59
N3915	NA	East of 18th Street	Flagler Avenue	1.92	2.11	2.29	2.60
N3920	B3920	Between 19th and 20th Street	Flagler Avenue	2.30	2.51	2.70	2.96

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N3930	B3930	20th Street	Duck to Eagle Avenue	2.62	2.75	2.86	3.12
N400	B400	Alberta Street	Seminole Avenue	3.51	3.55	3.60	3.69
N4000	B4000	Whitehead Street	Between Fleming and Southard Streets	5.96	6.82	7.01	7.32
N4010	B4010	Whitehead Street	Fleming Street	2.50	2.81	3.19	4.04
N410	B410	William Street	Washington Street	3.69	3.83	3.92	4.02
N4100	NA	Behind Shopping Center	Northside Drive	2.22	2.45	2.72	3.06
N4102	B4100	15th-ish	Northside Drive	2.30	2.55	2.83	3.19
N4105	B4105	16th-ish	West of Donald Avenue	2.32	2.57	2.85	3.20
N4110	B4110	17th Street	Donald area	2.36	2.60	2.88	3.23
N4115	B4115	16th Street	North of Donald Avenue	2.36	2.60	2.88	3.23
N4120	B4120	18th Street	Donald Avenue	2.36	2.60	2.88	3.23
N4125	B4125	20th Street	Northside Drive	2.31	2.47	2.63	2.96
N4130	B4130	20th Street	Donald Avenue	2.41	2.59	2.75	3.15
N4140	B4140	19th Street	Donald Avenue	2.45	2.68	2.91	3.23
N4143	NA	19th Street	Donald Avenue	2.36	2.61	2.89	3.22
N4145	B4145	19th Street	Cindy Avenue	2.82	2.88	2.93	3.20
N4147	B4147	18th Terrace	Donald Avenue	2.59	2.79	2.96	3.23
N4150	B4150	20th Street	Cindy Avenue	2.42	2.59	2.76	3.15

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N4160	B4160	18th Terrace	Northside Drive	2.45	2.66	2.81	3.00
N4170	B4170	18th Street	Northside Drive	2.52	2.79	3.00	3.20
N4175	B4175	17th Street	16th Terrace	2.81	2.95	3.04	3.16
N4180	B4180	16th Terrace	Northside Drive	2.36	2.60	2.88	3.23
N4200	B4200	Trinidad Drive	Venetian Street	2.23	2.27	2.32	2.40
N4210	B4210	Jamaica Drive	Venetian Street	1.75	1.89	1.96	2.04
N4220	B4220	Bahama Drive	Venetian Street	1.88	1.98	2.04	2.12
N500	B500	Duval Street	Boundary, past South Street	1.00	1.45	1.66	2.04
N5000	B5000	Whitehead Street	South Street	1.56	1.70	1.84	2.09
N510	B510	Simonton Street	South Street	4.18	4.23	4.27	4.35
N520	B520	Simonton Street	United Street	4.35	4.45	4.53	4.72
N530	B530	Simonton Street	Louisa Street	4.01	4.74	5.52	5.77
N540	B540	Duval Street	Catherine Street	3.21	3.37	3.45	3.62
N600	B600	Fort Street	Amelia Street	1.92	2.34	2.74	3.23
N6000	B6000	10th Street	Harris Avenue	2.32	2.51	2.76	2.96
N605	B605	Emma Street	Amelia Street	2.67	2.91	3.09	3.39
N610	B610	Emma Street	Virginia Street	2.88	3.03	3.17	3.43
N615	NA	Emma Street	Amelia Street	2.72	2.89	3.07	3.38

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N620	B620	Howe Street	Virginia Street	3.03	3.14	3.24	3.45
N625	B625	Whitehead Street	Catherine Street	2.73	2.89	3.07	3.37
N627	B627	Whitehead Street	Amelia Street	2.97	3.03	3.12	3.37
N628	B628	Whitehead Street	Virginia Street	3.81	3.82	3.86	3.94
N630	B630	Whitehead Street	United Street	2.70	2.83	2.96	3.23
N635	B635	Fort Street	Truman Avenue	3.58	3.74	3.86	4.06
N640	B640	Emma Street	Truman Avenue	3.65	3.75	3.85	4.04
N641	B641	Thomas Street	Truman Avenue	4.83	4.90	4.97	5.11
N642	B642	Emma Street	Olivia Street	3.75	3.88	4.01	4.21
N643	B643	Emma Street	Petronia Street	4.24	4.33	4.40	4.55
N645	B645	Thomas Street	Petronia Street	4.10	4.55	5.12	6.48
N700	B700	White Street	Fleming Street	2.56	2.62	2.68	2.89
N705	B705	Frances Street	Fleming Street	3.16	3.20	3.24	3.32
N710	B710	White Street	Southard Street	3.35	3.98	4.62	4.84
N720	B720	White Street	Angela Street	4.04	4.79	5.13	5.30
N730	B730	Ashe Street	Angela Street	4.34	4.82	4.96	5.10
N750	B750	Frances Street	Petronia Street	4.92	5.02	5.13	5.37
N755	B755	Frances Street	Olivia Street	4.94	5.09	5.23	5.42
N800	B800	Florida Street	Eliza Street	3.20	3.62	3.75	3.89

Table A-1. Simulated Peak Stage Results (NAVD 88) from the Existing Conditions Model of the City's Stormwater System

Node	Sub-basin	Road North/South Reference	Road East/West Reference	5-year, 24-hour	10-year, 24-hour	25-year, 72-hour	100-year, 72-hour
N810	B810	White Street	Eliza Street	5.00	5.10	5.16	5.22
N820	B820	White Street	Virginia Street	2.73	3.08	3.53	4.54
N830	B830	Varela Street	Virginia Street	5.09	5.15	5.19	5.29
N900	B900	Pearl Street	Albury Street	2.51	2.76	3.02	3.86
N905	B905	Pearl Street	Petronia Street	3.25	3.54	3.78	4.11
N920	B920	Florida Street	Newton Street	3.25	3.54	3.78	4.11

Note:

PS = pump station

Appendix B
Analysis of Tidal Data

This appendix was from the City of Key West Sea Level Policy (Jacobs 2021b), but the text was slightly edited for style and clarity.

Calculation of Updated MHHW Elevation

The nearest active tide gauge, operated by the National Oceanic and Atmospheric Administration (NOAA), to the City of Key West is Station No. 8724580, Key West, Florida (24°33.0 N, 81°48.5 W), where the available measured data of water level date back to January 1914 (Figure B-1).

Figure B-1. City of Key West NOAA Station No. 8724580, Key West, Florida (24°33.0 N, 81°48.5 W)

Table B-1 lists the published tidal datums at the station for the previous tidal epoch (1960 through 1978) and the present tidal epoch (1983 through 2001). As shown in Table B-1, there has been an increase in the datum elevation on the order of 0.2 foot across the board, assuming that the vertical elevation of the Station Datum, which is the absolute zero of the measuring tide gauge, remains unchanged.

It is then conceivable that this documented rise in mean higher high water (MHHW) may continue into the post-2001 period, and it is essential to account for this rise in MHHW from 2002 to 2020.

Table B-1. Published Tidal Datums, Key West, Florida

Tidal/Vertical Datum	Elevations (feet Station Datum)		
	Previous Tidal Epoch (1960–1978)	Present Tidal Epoch (1983–2001)	Difference (Previous minus Present)
MHHW	6.17	6.37	0.20
MHW	5.88	6.08	0.20
MTL	5.23	5.44	0.21
MSL	5.25	5.45	0.20
DTL	5.25	5.46	0.21
MLW	4.57	4.80	0.23
MLLW	4.33	4.56	0.23
NAVD 88	N/A	6.32	N/A
STND	0.00	0.00	0.00

Source: https://tidesandcurrents.noaa.gov/datums.html?id=8724580
Note: Station Datum conversion to NAVD 88 is -6.32 feet.
DTL = mean diurnal tide level
MHHW = mean higher-high water
MHW = mean high water
MLLW = mean lower low water
MLW = mean low water
MSL = mean sea level
MTL = mean tide level
NAVD 88 = North American Vertical Datum of 1988
STND = station datum (for the Key West NOAA tide gauge)

Purpose

The purpose of the assessment is to estimate the rise in MHHW from 2001 through the present that may be captured in the measured water level data by conducting a harmonic analysis of the measured time series to filter out the non-tidal components and calculating the resulting MHHW of the filtered time series that contains astronomical tide signals only.

Method 1 NOAA Data Review Post-2001

MHHW water levels for each month from 2002 to 2020 (referenced to NAVD 88) were extracted from the NOAA tides website[1]. The data were plotted (refer to Figure B-2) to show the linear

[1] https://tidesandcurrents.noaa.gov/waterlevels.html?id=8724580

trend of MHHW observed per month, with events predicted during the month of September to November highlighted.

Figure B-2 shows an approximate linearly increase trend that reaches a value of 0.4 foot NAVD 88 for year 2020 if all the epoch is included; and a seasonal high tide of 0.8 foot NAVD 88 in year 2020 if only the highest events during the months of September to November are used. The latest trend is a rise of 0.6 foot as compared to the 1983 to 2001 tidal epoch (0.2 foot NAVD 88).

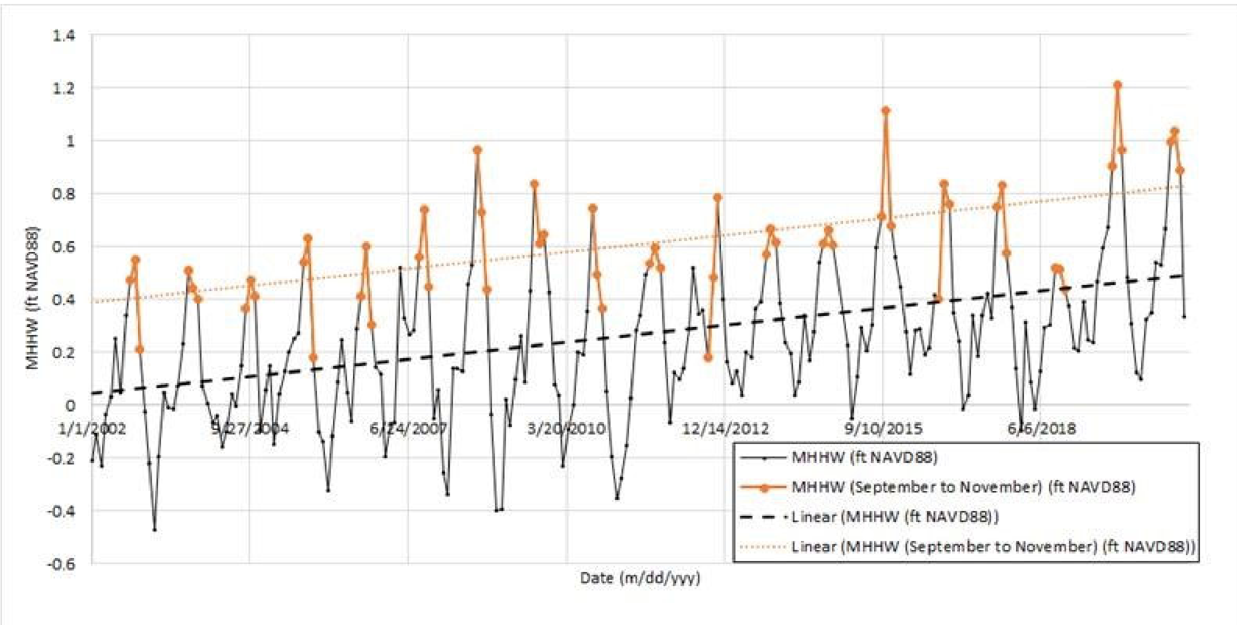

Figure B-2. MHHW Trend for 2002 to 2020 Epoch

Figure B-3 demonstrates an average high water elevation from 2015 to 2020 of approximately 0.8 foot, which aligns with the current trends analysis, resulting in an average high water elevation of 0.8 foot, as shown on Figure B-2.

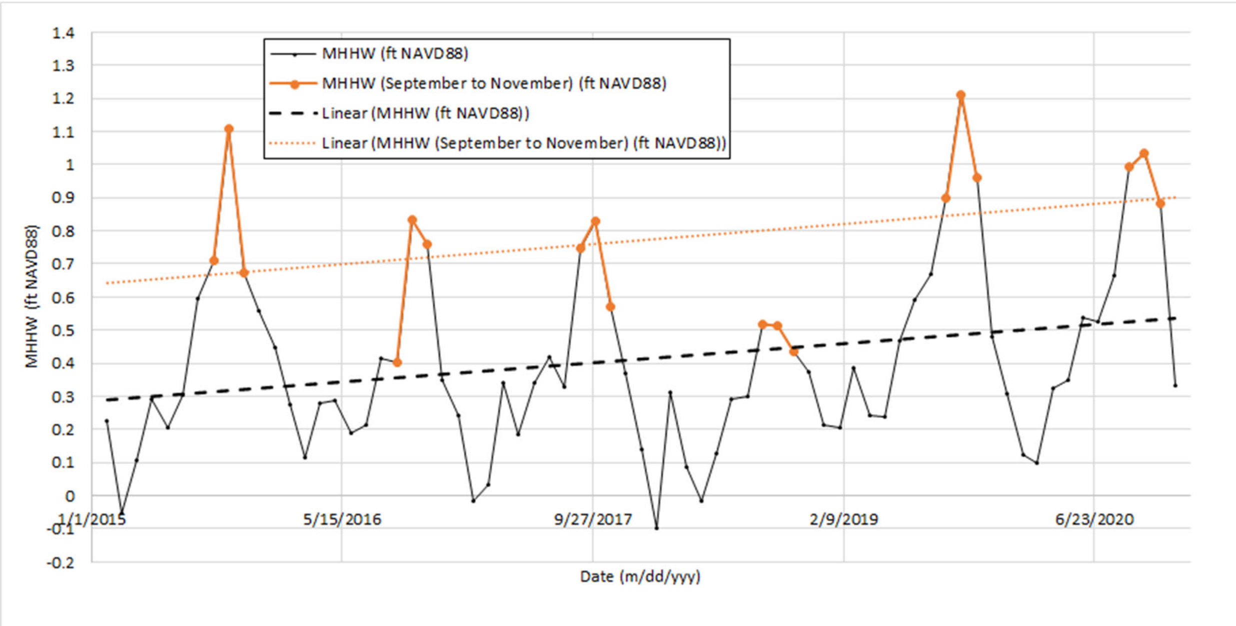

Figure B-3. MHHW 5-Year Trend Analysis for 2015 to 2020

Method 2 NOAA Calculator

In addition, the NOAA Tidal Analysis Datum Calculator[2] was used to calculate the updated MHHW, which serves as a check to Method 1. This method used the 19-year epoch of 2002 to 2020 and computed monthly means to derive tidal datum parameters. This analysis yields the same MHW of 0.4 foot NAVD 88 in 2020 and confirmed the approach described in Method 1.

Results and Recommendations

Figure B-2 presents an MHW of 0.4 foot NAVD 88 in year 2020 and a seasonal (September to November) MHHW of 0.8 foot NAVD 88. This figure suggests that the tides and MHHW are rising in step over the same time span. Therefore, Jacobs recommends that a seasonal high tide of 0.8 foot NAVD 88 be adopted and to use the revised 2020 results as the start year to calculate the sea level rise projections for policy considerations.

[2] https://access.co-ops.nos.noaa.gov/datumcalc/

Appendix C
Proposed Project Cost Opinions

Study Area 1A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 71,411.59		$ 71,411.59	
	Maintenance of Traffic	1.00	LS	$ 30,604.97		$ 30,604.97	
	Temporary Erosion Control Measures	1.00	LS	$ 10,201.66		$ 10,201.66	
	Activity SubTotal					$112,218	
	Proposed Conveyance Improvements						
	Pavement Replacement	1,566.67	SY	$ 71.00		$ 111,233.33	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	9.00	EA	$ 1,875.00		$ 16,875.00	
	Remove & Dispose of Existing Pipes	1,700.00	LF	$ 55.00		$ 93,500.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	10.00	CY	$ 203.75		$ 2,037.50	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	-	EA	$ 37,625.00		$ -	
	Concrete curb and gutter	1,175.00	LF	$ 46.00		$ 54,050.00	
	Sidewalk Replacement	5,875.00	SF	$ 12.00		$ 70,500.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	10.00	EA	$ 8,100.00		$ 81,000.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	3.00	EA	$ 11,100.00		$ 33,300.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	250.00	LF	$ 231.00		$ 57,750.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	260.00	LF	$ 315.00		$ 81,900.00	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	190.00	LF	$ 390.00		$ 74,100.00	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 8'-10' depth
	48" Stormwater Pipe	180.00	LF	$ 540.00		$ 97,200.00	Assume all pipe at 8'-10' depth
	54" Stormwater Pipe	1,020.00	LF	$ 615.00		$ 627,300.00	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	-	LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth
	Dewatering System Installation	1,900.00	LF	$ 31.25		$ 59,375.00	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	5.00	LS	$ 20,000.00		$ 100,000.00	Assume 1 per intersection

	Activity SubTotal					$1,693,661	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	-	EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well	-	EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	148.00	CY	$ 203.75		$ 30,155.00	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 172,178.00		$ 172,178.00	Assume 1 per pipe penetration
	Activity SubTotal					$346,670	
	Proposed Road Elevation Adjustments						
	Black Base		Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments		EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs		LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs		SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs		SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	
	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	

	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$2,152,549	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 322,882	
	Contractor Proffit			10%		$ 215,255	
	Engineering / Design			22%		$ 473,561	
	Contingency / Market Volatility			25%		$ 672,672	
	Total Including Contingencies					$3,836,919	

Study Area 1A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 208,177.85		$ 208,177.85	
	Maintenance of Traffic	1.00	LS	$ 89,219.08		$ 89,219.08	
	Temporary Erosion Control Measures	1.00	LS	$ 29,739.69		$ 29,739.69	
	Activity SubTotal					$327,137	
	Proposed Conveyance Improvements						
	Pavement Replacement	1,566.67	SY	$ 71.00		$ 111,233.33	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	9.00	EA	$ 1,875.00		$ 16,875.00	
	Remove & Dispose of Existing Pipes	1,700.00	LF	$ 55.00		$ 93,500.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	10.00	CY	$ 203.75		$ 2,037.50	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	-	EA	$ 37,625.00		$ -	
	Concrete curb and gutter	1,175.00	LF	$ 46.00		$ 54,050.00	
	Sidewalk Replacement	5,875.00	SF	$ 12.00		$ 70,500.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	10.00	EA	$ 8,100.00		$ 81,000.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	3.00	EA	$ 11,100.00		$ 33,300.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	250.00	LF	$ 231.00		$ 57,750.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	260.00	LF	$ 315.00		$ 81,900.00	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	190.00	LF	$ 390.00		$ 74,100.00	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 8'-10' depth
	48" Stormwater Pipe	180.00	LF	$ 540.00		$ 97,200.00	Assume all pipe at 8'-10' depth
	54" Stormwater Pipe	1,020.00	LF	$ 615.00		$ 627,300.00	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	-	LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth
	Dewatering System Installation	1,900.00	LF	$ 31.25		$ 59,375.00	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	5.00	LS	$ 20,000.00		$ 100,000.00	Assume 1 per intersection

	Activity SubTotal					$1,693,661	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	-	EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well	-	EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	148.00	CY	$ 203.75		$ 30,155.00	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 172,178.00		$ 172,178.00	Assume 1 per pipe penetration
	Activity SubTotal					$346,670	
	Proposed Road Elevation Adjustments						
	Black Base		Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments		EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs		LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs		SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs		SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						
	Screening Chamber	1.00	LS	$ 231,125.00		$ 231,125.00	
	Piping, Valves, Fittings	1.00	LS	$ 988,125.00		$ 988,125.00	
	Generator System	1.00	LS	$ 270,000.00		$ 270,000.00	
	Pump Station	1.00	LS	$ 1,482,750.00		$ 1,482,750.00	
	Electrical Platform & Stairs	1.00	LS	$ 149,000.00		$ 149,000.00	
	Electrical and Instrumentation and Control Work	1.00	LS	$ 489,982.50		$ 489,982.50	

	Landscaping / Screening Allowance	1.00	LS	$ 46,625.00		$ 46,625.00	
	Aesthetics Allowance	1.00	LS	$ 250,000.00		$ 250,000.00	
	Activity SubTotal					$3,907,608	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$6,275,075	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 941,261	
	Contractor Proffit			10%		$ 627,508	
	Engineering / Design			22%		$ 1,380,517	
	Contingency / Market Volatility			25%		$ 1,960,961	
	Total Including Contingencies					$11,185,322	

Study Area 1B Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 162,421.90		$ 162,421.90	
	Maintenance of Traffic	1.00	LS	$ 69,609.39		$ 69,609.39	
	Temporary Erosion Control Measures	1.00	LS	$ 23,203.13		$ 23,203.13	
	Activity SubTotal					$255,234	
	Proposed Conveyance Improvements						
	Pavement Replacement	3,400.00	SY	$ 71.00		$ 241,400.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	45.00	EA	$ 1,875.00		$ 84,375.00	
	Remove & Dispose of Existing Pipes	1,555.00	LF	$ 55.00		$ 85,525.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	92.00	CY	$ 203.75		$ 18,745.00	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	2.00	EA	$ 37,625.00		$ 75,250.00	
	Concrete curb and gutter	2,550.00	LF	$ 46.00		$ 117,300.00	
	Sidewalk Replacement	12,750.00	SF	$ 12.00		$ 153,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	7,680.00	SF	$ 12.00		$ 92,160.00	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	20.00	EA	$ 8,100.00		$ 162,000.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	6.00	EA	$ 11,100.00		$ 66,600.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe	1,880.00	LF	$ 393.75		$ 740,250.00	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	275.00	LF	$ 465.00		$ 127,875.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe	395.00	LF	$ 675.00		$ 266,625.00	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	2,550.00	LF	$ 31.25		$ 79,687.50	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	14.00	LS	$ 20,000.00		$ 280,000.00	Assume 1 per intersection
	Activity SubTotal					$2,724,333	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	3.00	EA	$ 1,875.00		$ 5,625.00	
	Grout and Seal Existing Well	3.00	EA	$ 80,000.00		$ 240,000.00	
	Activity SubTotal					$245,625	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	1.00	EA	$ 43,750.00		$ 43,750.00	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$532,443	
	Proposed Road Elevation Adjustments						
	Black Base	2,525.00	Ton	$ 257.00		$ 648,925.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	20.00	EA	$ 2,425.00		$ 48,500.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	5,800.00	LF	$ 46.00		$ 266,800.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	14,500.00	SF	$ 12.00		$ 174,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$1,138,225	
	Proposed Stormwater Pump Station						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	

	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	
	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$4,895,860	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 734,379	
	Contractor Proffit			10%		$ 489,586	
	Engineering / Design			22%		$ 1,077,089	
	Contingency / Market Volatility			25%		$ 1,529,956	
	Total Including Contingencies					$8,726,871	

Study Area 1B Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 435,954.43		$ 435,954.43	
	Maintenance of Traffic	1.00	LS	$ 186,837.61		$ 186,837.61	
	Temporary Erosion Control Measures	1.00	LS	$ 62,279.20		$ 62,279.20	
	Activity SubTotal					$685,071	
	Proposed Conveyance Improvements						
	Pavement Replacement	3,400.00	SY	$ 71.00		$ 241,400.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	45.00	EA	$ 1,875.00		$ 84,375.00	
	Remove & Dispose of Existing Pipes	1,555.00	LF	$ 55.00		$ 85,525.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	92.00	CY	$ 203.75		$ 18,745.00	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	2.00	EA	$ 37,625.00		$ 75,250.00	
	Concrete curb and gutter	2,550.00	LF	$ 46.00		$ 117,300.00	
	Sidewalk Replacement	12,750.00	SF	$ 12.00		$ 153,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	7,680.00	SF	$ 12.00		$ 92,160.00	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	20.00	EA	$ 8,100.00		$ 162,000.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	6.00	EA	$ 11,100.00		$ 66,600.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe	1,880.00	LF	$ 393.75		$ 740,250.00	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	275.00	LF	$ 465.00		$ 127,875.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe	395.00	LF	$ 675.00		$ 266,625.00	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	2,550.00	LF	$ 31.25		$ 79,687.50	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	14.00	LS	$ 20,000.00		$ 280,000.00	Assume 1 per intersection
	Activity SubTotal					$2,724,333	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	3.00	EA	$ 1,875.00		$ 5,625.00	
	Grout and Seal Existing Well	3.00	EA	$ 80,000.00		$ 240,000.00	
	Activity SubTotal					$245,625	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	1.00	EA	$ 43,750.00		$ 43,750.00	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$532,443	
	Proposed Road Elevation Adjustments						
	Black Base	2,525.00	Ton	$ 257.00		$ 648,925.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	20.00	EA	$ 2,425.00		$ 48,500.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	5,800.00	LF	$ 46.00		$ 266,800.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	14,500.00	SF	$ 12.00		$ 174,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$1,138,225	
	Proposed Stormwater Pump Station						
	Screening Chamber	2.00	LS	$ 231,125.00		$ 462,250.00	
	Piping, Valves, Fittings	2.00	LS	$ 988,125.00		$ 1,976,250.00	
	Generator System	2.00	LS	$ 270,000.00		$ 540,000.00	
	Pump Station	2.00	LS	$ 1,482,750.00		$ 2,965,500.00	

	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	
	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	2.00	LS	$ 250,000.00		$ 500,000.00	
	Activity SubTotal					$7,815,215	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$13,140,912	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,971,137	
	Contractor Proffit			10%		$ 1,314,091	
	Engineering / Design			22%		$ 2,891,001	
	Contingency / Market Volatility			25%		$ 4,106,535	
	Total Including Contingencies					$23,423,676	

Study Area 2A & 2B Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 152,367.12		$ 152,367.12	
	Maintenance of Traffic	1.00	LS	$ 65,300.19		$ 65,300.19	
	Temporary Erosion Control Measures	1.00	LS	$ 21,766.73		$ 21,766.73	
	Activity SubTotal					$239,434	

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	Proposed Conveyance Improvements						
	Pavement Replacement	4,693.33	SY	$ 71.00		$ 333,226.67	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	13.00	EA	$ 1,875.00		$ 24,375.00	
	Remove & Dispose of Existing Pipes	675.00	LF	$ 55.00		$ 37,125.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	25.00	CY	$ 203.75		$ 5,093.75	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves		EA	$ 37,625.00		$ -	
	Concrete curb and gutter	3,520.00	LF	$ 46.00		$ 161,920.00	
	Sidewalk Replacement	17,600.00	SF	$ 12.00		$ 211,200.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	18.00	EA	$ 8,100.00		$ 145,800.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	14.00	EA	$ 11,100.00		$ 155,400.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)		EA	$ 20,000.00		$ -	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	415.00	LF	$ 231.00		$ 95,865.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	475.00	LF	$ 315.00		$ 149,625.00	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	1,165.00	LF	$ 390.00		$ 454,350.00	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	915.00	LF	$ 465.00		$ 425,475.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	550.00	LF	$ 690.00		$ 379,500.00	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	3,520.00	LF	$ 31.25		$ 110,000.00	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	9.00	LS	$ 20,000.00		$ 180,000.00	Assume 1 per intersection
	Activity SubTotal					$2,976,036	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	10.00	EA	$ 1,875.00		$ 18,750.00	
	Grout and Seal Existing Well	10.00	EA	$ 80,000.00		$ 800,000.00	
	Activity SubTotal					$818,750	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 172,178.00		$ 172,178.00	Assume 1 per pipe penetration
	Activity SubTotal					$316,515	
	Proposed Road Elevation Adjustments						
	Black Base	435.00	Ton	$ 257.00		$ 111,795.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	10.00	EA	$ 2,425.00		$ 24,250.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	1,000.00	LF	$ 46.00		$ 46,000.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	5,000.00	SF	$ 12.00		$ 60,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$242,045	
	Proposed Stormwater Pump Station						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	

	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	
	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$4,592,780	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 688,917	
	Contractor Proffit			10%		$ 459,278	
	Engineering / Design			22%		$ 1,010,412	
	Contingency / Market Volatility			25%		$ 1,435,244	
	Total Including Contingencies					$8,186,631	

Study Area 2A & 2B Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 425,899.64		$ 425,899.64	
	Maintenance of Traffic	1.00	LS	$ 182,528.42		$ 182,528.42	
	Temporary Erosion Control Measures	1.00	LS	$ 60,842.81		$ 60,842.81	
	Activity SubTotal					$669,271	
	Proposed Conveyance Improvements						
	Pavement Replacement	4,693.33	SY	$ 71.00		$ 333,226.67	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	13.00	EA	$ 1,875.00		$ 24,375.00	
	Remove & Dispose of Existing Pipes	675.00	LF	$ 55.00		$ 37,125.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	25.00	CY	$ 203.75		$ 5,093.75	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves		EA	$ 37,625.00		$ -	
	Concrete curb and gutter	3,520.00	LF	$ 46.00		$ 161,920.00	
	Sidewalk Replacement	17,600.00	SF	$ 12.00		$ 211,200.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	18.00	EA	$ 8,100.00		$ 145,800.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	14.00	EA	$ 11,100.00		$ 155,400.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)		EA	$ 20,000.00		$ -	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	415.00	LF	$ 231.00		$ 95,865.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	475.00	LF	$ 315.00		$ 149,625.00	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	1,165.00	LF	$ 390.00		$ 454,350.00	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	915.00	LF	$ 465.00		$ 425,475.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	550.00	LF	$ 690.00		$ 379,500.00	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	3,520.00	LF	$	31.25		$	110,000.00	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$	17,846.84		$	107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	9.00	LS	$	20,000.00		$	180,000.00	Assume 1 per intersection
	Activity SubTotal							$2,976,036	
	Plug Existing Gravity Wells								
	Demo Existing Well to 4' Below Pavement	10.00	EA	$	1,875.00		$	18,750.00	
	Grout and Seal Existing Well	10.00	EA	$	80,000.00		$	800,000.00	
	Activity SubTotal							$818,750	
	Outfall Improvements								
	Remove / Abandon Existing Outfall	-	CY	$	203.75		$	-	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$	28.44		$	63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$	17,846.84		$	17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$	43,750.00		$	-	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$	62,500.00		$	62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$	114,785.00		$	-	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$	172,178.00		$	172,178.00	Assume 1 per pipe penetration
	Activity SubTotal							$316,515	
	Proposed Road Elevation Adjustments								
	Black Base	435.00	Ton	$	257.00		$	111,795.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	10.00	EA	$	2,425.00		$	24,250.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	1,000.00	LF	$	46.00		$	46,000.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	5,000.00	SF	$	12.00		$	60,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$	12.00		$	-	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal							$242,045	
	Proposed Stormwater Pump Station								
	Screening Chamber	2.00	LS	$	231,125.00		$	462,250.00	
	Piping, Valves, Fittings	2.00	LS	$	988,125.00		$	1,976,250.00	
	Generator System	2.00	LS	$	270,000.00		$	540,000.00	
	Pump Station	2.00	LS	$	1,482,750.00		$	2,965,500.00	

	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	
	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	2.00	LS	$ 250,000.00		$ 500,000.00	
	Activity SubTotal					$7,815,215	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$12,837,832	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,925,675	
	Contractor Proffit			10%		$ 1,283,783	
	Engineering / Design			22%		$ 2,824,323	
	Contingency / Market Volatility			25%		$ 4,011,823	
	Total Including Contingencies					$22,883,436	

Study Area 2C Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 37,083.69		$ 37,083.69	
	Maintenance of Traffic	1.00	LS	$ 15,893.01		$ 15,893.01	
	Temporary Erosion Control Measures	1.00	LS	$ 5,297.67		$ 5,297.67	
	Activity SubTotal					$58,274	
	Proposed Conveyance Improvements						
	Pavement Replacement	-	SY	$ 71.00		$ -	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	-	EA	$ 1,875.00		$ -	
	Abandon / Plug Existing Pipe	-	CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	-	EA	$ 37,625.00		$ -	
	Concrete curb and gutter	-	LF	$ 46.00		$ -	
	Sidewalk Replacement	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	-	EA	$ 8,100.00		$ -	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	-	EA	$ 11,100.00		$ -	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	-	LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 8'-10' depth
	48" Stormwater Pipe	-	LF	$ 540.00		$ -	Assume all pipe at 8'-10' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	-	LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth
	Dewatering System Installation	2,600.00	LF	$ 31.25		$ 81,250.00	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	6.00	LS	$ 20,000.00		$ 120,000.00	Assume 1 per intersection
	Activity SubTotal					$254,791	

	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	-	EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well	-	EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	-	SF	$ 28.44		$ -	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	-	Months	$ 17,846.84		$ -	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	-	EA	$ 62,500.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	-	EA	$ 172,178.00		$ -	Assume 1 per pipe penetration
	Activity SubTotal					$0	
	Proposed Road Elevation Adjustments						
	Black Base	1,435.50	Ton	$ 257.00		$ 368,923.50	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	12.00	EA	$ 2,425.00		$ 29,100.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	5,200.00	LF	$ 46.00		$ 239,200.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	13,000.00	SF	$ 12.00		$ 156,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	960.00	SF	$ 12.00		$ 11,520.00	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$804,744	
	Proposed Stormwater Pump Station						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	
	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	
	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	

	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$1,117,808	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 167,671	
	Contractor Proffit			10%		$ 111,781	
	Engineering / Design			22%		$ 245,918	
	Contingency / Market Volatility			25%		$ 349,315	
	Total Including Contingencies					$1,992,493	

Study Area 3A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 80,150.45		$ 80,150.45	
	Maintenance of Traffic	1.00	LS	$ 34,350.19		$ 34,350.19	
	Temporary Erosion Control Measures	1.00	LS	$ 11,450.06		$ 11,450.06	
	Activity SubTotal					$125,951	

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	Proposed Conveyance Improvements						
	Pavement Replacement	960.00	SY	$ 71.00		$ 68,160.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	-	EA	$ 1,875.00		$ -	
	Abandon / Plug Existing Pipe	-	CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	2.00	EA	$ 37,625.00		$ 75,250.00	
	Concrete curb and gutter	-	LF	$ 46.00		$ -	
	Sidewalk Replacement	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	4.00	EA	$ 8,100.00		$ 32,400.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	-	EA	$ 11,100.00		$ -	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe	360.00	LF	$ 675.00		$ 243,000.00	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	Dewatering System Installation	360.00	LF	$ 31.25		$ 11,250.00	Estimate LF of both minor and major pipe sizes

	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	4.00	LS	$ 20,000.00		$ 80,000.00	Assume 1 per intersection
	Activity SubTotal					$643,601	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	1.00	EA	$ 1,875.00		$ 1,875.00	
	Grout and Seal Existing Well	1.00	EA	$ 80,000.00		$ 80,000.00	
	Activity SubTotal					$81,875	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	4,500.00	SF	$ 28.44		$ 127,980.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	4.00	Months	$ 17,846.84		$ 71,387.35	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$324,367	
	Proposed Road Elevation Adjustments						
	Black Base	2,600.00	Ton	$ 257.00		$ 668,200.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	18.00	EA	$ 2,425.00		$ 43,650.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	7,280.00	LF	$ 46.00		$ 334,880.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	3,640.00	SF	$ 12.00		$ 43,680.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	12,480.00	SF	$ 12.00		$ 149,760.00	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$1,240,170	
	Proposed Stormwater Pump Station						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	

	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	
	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$2,415,964	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 362,395	
	Contractor Proffit			10%		$ 241,596	
	Engineering / Design			22%		$ 531,512	
	Contingency / Market Volatility			25%		$ 754,989	
	Total Including Contingencies					$4,306,455	

Study Area 3A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 332,919.23		$ 332,919.23	
	Maintenance of Traffic	1.00	LS	$ 142,679.67		$ 142,679.67	
	Temporary Erosion Control Measures	1.00	LS	$ 47,559.89		$ 47,559.89	
	Activity SubTotal					$523,159	
	Proposed Conveyance Improvements						
	Pavement Replacement	960.00	SY	$ 71.00		$ 68,160.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	-	EA	$ 1,875.00		$ -	
	Abandon / Plug Existing Pipe	-	CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	2.00	EA	$ 37,625.00		$ 75,250.00	
	Concrete curb and gutter	-	LF	$ 46.00		$ -	
	Sidewalk Replacement	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	4.00	EA	$ 8,100.00		$ 32,400.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	-	EA	$ 11,100.00		$ -	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe	360.00	LF	$ 675.00		$ 243,000.00	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	Dewatering System Installation	360.00	LF	$ 31.25		$ 11,250.00	Estimate LF of both minor and major pipe sizes

	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	4.00	LS	$ 20,000.00		$ 80,000.00	Assume 1 per intersection
	Activity SubTotal					$643,601	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	1.00	EA	$ 1,875.00		$ 1,875.00	
	Grout and Seal Existing Well	1.00	EA	$ 80,000.00		$ 80,000.00	
	Activity SubTotal					$81,875	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	4,500.00	SF	$ 28.44		$ 127,980.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	4.00	Months	$ 17,846.84		$ 71,387.35	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$324,367	
	Proposed Road Elevation Adjustments						
	Black Base	2,600.00	Ton	$ 257.00		$ 668,200.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	18.00	EA	$ 2,425.00		$ 43,650.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	7,280.00	LF	$ 46.00		$ 334,880.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	3,640.00	SF	$ 12.00		$ 43,680.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	12,480.00	SF	$ 12.00		$ 149,760.00	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$1,240,170	
	Proposed Stormwater Pump Station						
	Screening Chamber	2.00	LS	$ 231,125.00		$ 462,250.00	
	Piping, Valves, Fittings	2.00	LS	$ 988,125.00		$ 1,976,250.00	
	Generator System	2.00	LS	$ 270,000.00		$ 540,000.00	
	Pump Station	2.00	LS	$ 1,482,750.00		$ 2,965,500.00	

	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	
	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	2.00	LS	$ 250,000.00		$ 500,000.00	
	Activity SubTotal					$7,221,965	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$10,035,137	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,505,270	
	Contractor Proffit			10%		$ 1,003,514	
	Engineering / Design			22%		$ 2,207,730	
	Contingency / Market Volatility			25%		$ 3,135,980	
	Total Including Contingencies					$17,887,631	

Study Area 4A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 88,969.49		$ 88,969.49	
	Maintenance of Traffic	1.00	LS	$ 38,129.78		$ 38,129.78	
	Temporary Erosion Control Measures	1.00	LS	$ 12,709.93		$ 12,709.93	
	Activity SubTotal					$139,809	
	Proposed Conveyance Improvements						
	Pavement Replacement	2,473.33	SY	$ 71.00		$ 175,606.67	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	8.00	EA	$ 1,875.00		$ 15,000.00	
	Remove & Dispose of Existing Pipes		LF	$ 55.00		$ -	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe		CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	1.00	EA	$ 37,625.00		$ 37,625.00	
	Concrete curb and gutter	1,855.00	LF	$ 46.00		$ 85,330.00	
	Sidewalk Replacement		SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	6.00	EA	$ 8,100.00		$ 48,600.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	8.00	EA	$ 11,100.00		$ 88,800.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)		EA	$ 20,000.00		$ -	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe		LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe		LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	42" Elliptical Stormwater Pipe	1,680.00	LF	$ 581.25		$ 976,500.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe		LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth

	54" Elliptical Stormwater Pipe	175.00	LF	$ 768.75		$ 134,531.25	Assume all pipe at 8'-10' depth for pump station outfall
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	Dewatering System Installation	1,855.00	LF	$ 31.25		$ 57,968.75	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	5.00	LS	$ 20,000.00		$ 100,000.00	Assume 1 per intersection
	Activity SubTotal					$1,827,043	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	2.00	EA	$ 1,875.00		$ 3,750.00	
	Grout and Seal Existing Well	2.00	EA	$ 80,000.00		$ 160,000.00	
	Activity SubTotal					$163,750	
	Outfall Improvements						
	Remove / Abandon Existing Outfall		CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$551,193	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						
	Screening Chamber		LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings		LS	$ 988,125.00		$ -	

	Generator System		LS	$ 270,000.00		$ -	
	Pump Station		LS	$ 1,482,750.00		$ -	
	Electrical Platform & Stairs		LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work		LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance		LS	$ 46,625.00		$ -	
	Aesthetics Allowance		LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$2,681,795	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 402,269	
	Contractor Proffit			10%		$ 268,179	
	Engineering / Design			22%		$ 589,995	
	Contingency / Market Volatility			25%		$ 838,061	
	Total Including Contingencies					$4,780,299	

Study Area 4A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 362,502.02		$ 362,502.02	
	Maintenance of Traffic	1.00	LS	$ 155,358.01		$ 155,358.01	
	Temporary Erosion Control Measures	1.00	LS	$ 51,786.00		$ 51,786.00	
	Activity SubTotal					$569,646	
	Proposed Conveyance Improvements						
	Pavement Replacement	2,473.33	SY	$ 71.00		$ 175,606.67	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	8.00	EA	$ 1,875.00		$ 15,000.00	
	Remove & Dispose of Existing Pipes		LF	$ 55.00		$ -	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe		CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	1.00	EA	$ 37,625.00		$ 37,625.00	
	Concrete curb and gutter	1,855.00	LF	$ 46.00		$ 85,330.00	
	Sidewalk Replacement		SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	6.00	EA	$ 8,100.00		$ 48,600.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	8.00	EA	$ 11,100.00		$ 88,800.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)		EA	$ 20,000.00		$ -	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe		LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe		LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	42" Elliptical Stormwater Pipe	1,680.00	LF	$ 581.25		$ 976,500.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe		LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth

	54" Elliptical Stormwater Pipe	175.00	LF	$ 768.75		$ 134,531.25	Assume all pipe at 8'-10' depth for pump station outfall
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	Dewatering System Installation	1,855.00	LF	$ 31.25		$ 57,968.75	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	5.00	LS	$ 20,000.00		$ 100,000.00	Assume 1 per intersection
	Activity SubTotal					$1,827,043	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	2.00	EA	$ 1,875.00		$ 3,750.00	
	Grout and Seal Existing Well	2.00	EA	$ 80,000.00		$ 160,000.00	
	Activity SubTotal					$163,750	
	Outfall Improvements						
	Remove / Abandon Existing Outfall		CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$551,193	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						
	Screening Chamber	2.00	LS	$ 231,125.00		$ 462,250.00	
	Piping, Valves, Fittings	2.00	LS	$ 988,125.00		$ 1,976,250.00	

	Generator System	2.00	LS	$ 270,000.00		$ 540,000.00	
	Pump Station	2.00	LS	$ 1,482,750.00		$ 2,965,500.00	
	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	
	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	2.00	LS	$ 250,000.00		$ 500,000.00	
	Activity SubTotal					$7,815,215	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$10,926,847	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,639,027	
	Contractor Proffit			10%		$ 1,092,685	
	Engineering / Design			22%		$ 2,403,906	
	Contingency / Market Volatility			25%		$ 3,414,640	
	Total Including Contingencies					$19,477,104	

Study Area 4B Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 124,742.86		$ 124,742.86	
	Maintenance of Traffic	1.00	LS	$ 53,461.23		$ 53,461.23	
	Temporary Erosion Control Measures	1.00	LS	$ 17,820.41		$ 17,820.41	
	Activity SubTotal					$196,024	
	Proposed Conveyance Improvements						
	Pavement Replacement	3,293.33	SY	$ 71.00		$ 233,826.67	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	19.00	EA	$ 1,875.00		$ 35,625.00	
	Remove & Dispose of Existing Pipes	3,150.00	LF	$ 55.00		$ 173,250.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe		CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	1.00	EA	$ 37,625.00		$ 37,625.00	
	Concrete curb and gutter	3,150.00	LF	$ 46.00		$ 144,900.00	
	Sidewalk Replacement	4,800.00	SF	$ 12.00		$ 57,600.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	18.00	EA	$ 8,100.00		$ 145,800.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)		EA	$ 11,100.00		$ -	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	12.00	EA	$ 20,000.00		$ 240,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe		LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe		LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	42" Elliptical Stormwater Pipe		LF	$ 581.25		$ -	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe	2,870.00	LF	$ 540.00		$ 1,549,800.00	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe		LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth

	54" Elliptical Stormwater Pipe		LF	$ 768.75		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	Dewatering System Installation	2,870.00	LF	$ 31.25		$ 89,687.50	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	8.00	LS	$ 20,000.00		$ 160,000.00	Assume 1 per intersection
	Activity SubTotal					$2,975,195	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement		EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well		EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	185.00	CY	$ 203.75		$ 37,693.75	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$588,887	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						
	Screening Chamber		LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings		LS	$ 988,125.00		$ -	

	Generator System		LS	$ 270,000.00		$ -	
	Pump Station		LS	$ 1,482,750.00		$ -	
	Electrical Platform & Stairs		LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work		LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance		LS	$ 46,625.00		$ -	
	Aesthetics Allowance		LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$3,760,106	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 564,016	
	Contractor Proffit			10%		$ 376,011	
	Engineering / Design			22%		$ 827,223	
	Contingency / Market Volatility			25%		$ 1,175,033	
	Total Including Contingencies					$6,702,389	

Study Area 4B Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 398,275.39		$ 398,275.39	
	Maintenance of Traffic	1.00	LS	$ 170,689.45		$ 170,689.45	
	Temporary Erosion Control Measures	1.00	LS	$ 56,896.48		$ 56,896.48	
	Activity SubTotal					$625,861	

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	Proposed Conveyance Improvements						
	Pavement Replacement	3,293.33	SY	$ 71.00		$ 233,826.67	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	19.00	EA	$ 1,875.00		$ 35,625.00	
	Remove & Dispose of Existing Pipes	3,150.00	LF	$ 55.00		$ 173,250.00	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe		CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	1.00	EA	$ 37,625.00		$ 37,625.00	
	Concrete curb and gutter	3,150.00	LF	$ 46.00		$ 144,900.00	
	Sidewalk Replacement	4,800.00	SF	$ 12.00		$ 57,600.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	18.00	EA	$ 8,100.00		$ 145,800.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)		EA	$ 11,100.00		$ -	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	12.00	EA	$ 20,000.00		$ 240,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe		LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe		LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe		LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	42" Elliptical Stormwater Pipe		LF	$ 581.25		$ -	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe	2,870.00	LF	$ 540.00		$ 1,549,800.00	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe		LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth

	54" Elliptical Stormwater Pipe		LF	$ 768.75		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall
	Dewatering System Installation	2,870.00	LF	$ 31.25		$ 89,687.50	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	8.00	LS	$ 20,000.00		$ 160,000.00	Assume 1 per intersection
	Activity SubTotal					$2,975,195	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement		EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well		EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	185.00	CY	$ 203.75		$ 37,693.75	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	2.00	EA	$ 172,178.00		$ 344,356.00	Assume 1 per pipe penetration
	Activity SubTotal					$588,887	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						
	Screening Chamber	2.00	LS	$ 231,125.00		$ 462,250.00	
	Piping, Valves, Fittings	2.00	LS	$ 988,125.00		$ 1,976,250.00	

	Generator System	2.00	LS	$ 270,000.00		$ 540,000.00	
	Pump Station	2.00	LS	$ 1,482,750.00		$ 2,965,500.00	
	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	
	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	2.00	LS	$ 250,000.00		$ 500,000.00	
	Activity SubTotal					$7,815,215	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$12,005,158	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,800,774	
	Contractor Proffit			10%		$ 1,200,516	
	Engineering / Design			22%		$ 2,641,135	
	Contingency / Market Volatility			25%		$ 3,751,612	
	Total Including Contingencies					$21,399,194	

Study Area 4C Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 94,539.51		$ 94,539.51	
	Maintenance of Traffic	1.00	LS	$ 40,516.93		$ 40,516.93	
	Temporary Erosion Control Measures	1.00	LS	$ 13,505.64		$ 13,505.64	
	Activity SubTotal					$148,562	
	Proposed Conveyance Improvements						
	Pavement Replacement	1,850.00	SY	$ 71.00		$ 131,350.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	16.00	EA	$ 1,875.00		$ 30,000.00	
	Abandon / Plug Existing Pipe	4.00	CY	$ 203.75		$ 815.00	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	-	EA	$ 37,625.00		$ -	
	Concrete curb and gutter	575.00	LF	$ 46.00		$ 26,450.00	
	Sidewalk Replacement	2,875.00	SF	$ 12.00		$ 34,500.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Remove & Dispose of Existing Pipes	1,140.00	LF	$ 55.00		$ 62,700.00	Assume 1 CY per every 3.75 LF
	Inlets / Manholes (12" - 24" pipe connections)	16.00	EA	$ 8,100.00		$ 129,600.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner at each side of the road
	Inlets / Manholes (>24" pipe connections)	6.00	EA	$ 11,100.00		$ 66,600.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	320.00	LF	$ 186.00		$ 59,520.00	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe	260.00	LF	$ 393.75		$ 102,375.00	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	360.00	LF	$ 390.00		$ 140,400.00	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe	775.00	LF	$ 487.50		$ 377,812.50	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe	290.00	LF	$ 675.00		$ 195,750.00	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	250.00	LF	$ 690.00		$ 172,500.00	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	2,255.00	LF	$ 31.25		$ 70,468.75	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	6.00	LS	$ 20,000.00		$ 120,000.00	Assume 1 per intersection
	Activity SubTotal					$1,907,922	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	-	EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well	-	EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	4,500.00	SF	$ 28.44		$ 127,980.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	2.00	Months	$ 17,846.84		$ 35,693.68	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 172,178.00		$ 172,178.00	Assume 1 per pipe penetration
	Activity SubTotal					$460,852	
	Proposed Road Elevation Adjustments						
	Black Base	635.00	Ton	$ 257.00		$ 163,195.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	24.00	EA	$ 2,425.00		$ 58,200.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	1,460.00	LF	$ 46.00		$ 67,160.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	3,650.00	SF	$ 12.00		$ 43,800.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$332,355	
	Proposed Stormwater Pump Station						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	

	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	
	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$2,849,691	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 427,454	
	Contractor Proffit			10%		$ 284,969	
	Engineering / Design			22%		$ 626,932	
	Contingency / Market Volatility			25%		$ 890,528	
	Total Including Contingencies					$5,079,574	

Study Area 4C Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 368,072.04		$ 368,072.04	
	Maintenance of Traffic	1.00	LS	$ 157,745.16		$ 157,745.16	
	Temporary Erosion Control Measures	1.00	LS	$ 52,581.72		$ 52,581.72	
	Activity SubTotal					$578,399	
	Proposed Conveyance Improvements						
	Pavement Replacement	1,850.00	SY	$ 71.00		$ 131,350.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	16.00	EA	$ 1,875.00		$ 30,000.00	
	Abandon / Plug Existing Pipe	4.00	CY	$ 203.75		$ 815.00	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	-	EA	$ 37,625.00		$ -	
	Concrete curb and gutter	575.00	LF	$ 46.00		$ 26,450.00	
	Sidewalk Replacement	2,875.00	SF	$ 12.00		$ 34,500.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Remove & Dispose of Existing Pipes	1,140.00	LF	$ 55.00		$ 62,700.00	Assume 1 CY per every 3.75 LF
	Inlets / Manholes (12" - 24" pipe connections)	16.00	EA	$ 8,100.00		$ 129,600.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner at each side of the road
	Inlets / Manholes (>24" pipe connections)	6.00	EA	$ 11,100.00		$ 66,600.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)	4.00	EA	$ 20,000.00		$ 80,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	320.00	LF	$ 186.00		$ 59,520.00	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe	260.00	LF	$ 393.75		$ 102,375.00	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	360.00	LF	$ 390.00		$ 140,400.00	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe	775.00	LF	$ 487.50		$ 377,812.50	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe	290.00	LF	$ 675.00		$ 195,750.00	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	250.00	LF	$ 690.00		$ 172,500.00	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	2,255.00	LF	$ 31.25		$ 70,468.75	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	6.00	LS	$ 20,000.00		$ 120,000.00	Assume 1 per intersection
	Activity SubTotal					$1,907,922	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	-	EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well	-	EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	4,500.00	SF	$ 28.44		$ 127,980.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	2.00	Months	$ 17,846.84		$ 35,693.68	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	2.00	EA	$ 62,500.00		$ 125,000.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 172,178.00		$ 172,178.00	Assume 1 per pipe penetration
	Activity SubTotal					$460,852	
	Proposed Road Elevation Adjustments						
	Black Base	635.00	Ton	$ 257.00		$ 163,195.00	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	24.00	EA	$ 2,425.00		$ 58,200.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	1,460.00	LF	$ 46.00		$ 67,160.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	3,650.00	SF	$ 12.00		$ 43,800.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$332,355	
	Proposed Stormwater Pump Station						
	Screening Chamber	2.00	LS	$ 231,125.00		$ 462,250.00	
	Piping, Valves, Fittings	2.00	LS	$ 988,125.00		$ 1,976,250.00	
	Generator System	2.00	LS	$ 270,000.00		$ 540,000.00	
	Pump Station	2.00	LS	$ 1,482,750.00		$ 2,965,500.00	

	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	
	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	2.00	LS	$ 250,000.00		$ 500,000.00	
	Activity SubTotal					$7,815,215	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$11,094,743	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$1,664,211	
	Contractor Proffit			10%		$1,109,474	
	Engineering / Design			22%		$2,440,843	
	Contingency / Market Volatility			25%		$3,467,107	
	Total Including Contingencies					$19,776,379	

Study Area 15 Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 117,083.20		$ 117,083.20	
	Maintenance of Traffic	1.00	LS	$ 50,178.51		$ 50,178.51	
	Temporary Erosion Control Measures	1.00	LS	$ 16,726.17		$ 16,726.17	
	Activity SubTotal					$183,988	

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	Proposed Conveyance Improvements						
	Pavement Replacement	3,875.00	SY	$ 71.00		$ 275,125.00	Quantity assumes 24' wide trench width over LF of pipe installation outside of channel section
	Demo existing manhole to 4' below grade and backfill	-	EA	$ 1,875.00		$ -	
	Abandon / Plug Existing Pipe	4.00	CY	$ 203.75		$ 815.00	Assumes plugging the connection at Northside Drive
	Flap Gate / Check Valves	-	EA	$ 37,625.00		$ -	
	Concrete curb and gutter	1,450.00	LF	$ 46.00		$ 66,700.00	Assumes curb and gutter for the 1450 LF of 72" outfall pipe
	Sidewalk Replacement	3,625.00	SF	$ 12.00		$ 43,500.00	Assume 5' concrete wide sidewalk, 6" thick, all areas along half of the 1,450 LF of 72" outfall pipe run
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	4.00	EA	$ 8,100.00		$ 32,400.00	1 per 250 LF of the 900 LF of pipe enclosing the existing channel modified inlets to sit atop the pipe structure
	Inlets / Manholes (>24" pipe connections)	1.00	EA	$ 11,100.00		$ 11,100.00	1 new inlet / manhole at northside drive to disconnect from the channel run
	Inlets / Manholes (>48" pipe connections)	6.00	EA	$ 20,000.00		$ 120,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	-	LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 8'-10' depth
	48" Stormwater Pipe	-	LF	$ 540.00		$ -	Assume all pipe at 8'-10' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	-	LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth
	72" Stormwater Pipe	2,350.00	LF	$ 765.00		$ 1,797,750.00	Assume all pipe at 8'-10' depth
	Dewatering System Installation	2,350.00	LF	$ 31.25		$ 73,437.50	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	6.00	LS	$ 20,000.00		$ 120,000.00	Assume 1 per intersection

	Activity SubTotal					$2,647,909	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	3.00	EA	$ 1,875.00		$ 5,625.00	3 in vicinity of Donald Avenue
	Grout and Seal Existing Well	3.00	EA	$ 80,000.00		$ 240,000.00	3 in vicinity of Donald Avenue
	Activity SubTotal					$245,625	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	2.00	Months	$ 17,846.84		$ 35,693.68	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	-	EA	$ 62,500.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 48"	1.00	EA	$ 93,750.00		$ 93,750.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	-	EA	$ 172,178.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 258,267.00		$ 258,267.00	Assume 1 per pipe penetration
	Activity SubTotal					$451,701	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						150 CFS pump station assumed at 2x cost of 50 CFS Example
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	

	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	
	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Class 5 Construction Estimate of Probable Cost Construction Subtotal (no Pump Station)					$3,529,222	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 529,383.31	
	Contractor Proffit			10%		$ 352,922.21	
	Engineering / Design			22%		$ 776,428.86	
	Contingency / Market Volatility			25%		$ 1,102,881.90	
	Total Including Contingencies					$6,290,838	

Study Area 4D Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 390,615.72		$ 390,615.72	
	Maintenance of Traffic	1.00	LS	$ 167,406.74		$ 167,406.74	
	Temporary Erosion Control Measures	1.00	LS	$ 55,802.25		$ 55,802.25	
	Activity SubTotal					$613,825	

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	Proposed Conveyance Improvements						
	Pavement Replacement	3,875.00	SY	$ 71.00		$ 275,125.00	Quantity assumes 24' wide trench width over LF of pipe installation outside of channel section
	Demo existing manhole to 4' below grade and backfill	-	EA	$ 1,875.00		$ -	
	Abandon / Plug Existing Pipe	4.00	CY	$ 203.75		$ 815.00	Assumes plugging the connection at Northside Drive
	Flap Gate / Check Valves	-	EA	$ 37,625.00		$ -	
	Concrete curb and gutter	1,450.00	LF	$ 46.00		$ 66,700.00	Assumes curb and gutter for the 1450 LF of 72" outfall pipe
	Sidewalk Replacement	3,625.00	SF	$ 12.00		$ 43,500.00	Assume 5' concrete wide sidewalk, 6" thick, all areas along half of the 1,450 LF of 72" outfall pipe run
	Driveway Replacement	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	4.00	EA	$ 8,100.00		$ 32,400.00	1 per 250 LF of the 900 LF of pipe enclosing the existing channel modified inlets to sit atop the pipe structure
	Inlets / Manholes (>24" pipe connections)	1.00	EA	$ 11,100.00		$ 11,100.00	1 new inlet / manhole at northside drive to disconnect from the channel run
	Inlets / Manholes (>48" pipe connections)	6.00	EA	$ 20,000.00		$ 120,000.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	-	LF	$ 231.00		$ -	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	-	LF	$ 390.00		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 8'-10' depth
	48" Stormwater Pipe	-	LF	$ 540.00		$ -	Assume all pipe at 8'-10' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	-	LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth
	72" Stormwater Pipe	2,350.00	LF	$ 765.00		$ 1,797,750.00	Assume all pipe at 8'-10' depth
	Dewatering System Installation	2,350.00	LF	$ 31.25		$ 73,437.50	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	6.00	LS	$ 20,000.00		$ 120,000.00	Assume 1 per intersection

	Activity SubTotal					$2,647,909	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	3.00	EA	$ 1,875.00		$ 5,625.00	3 in vicinity of Donald Avenue
	Grout and Seal Existing Well	3.00	EA	$ 80,000.00		$ 240,000.00	3 in vicinity of Donald Avenue
	Activity SubTotal					$245,625	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	2.00	Months	$ 17,846.84		$ 35,693.68	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	-	EA	$ 62,500.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 48"	1.00	EA	$ 93,750.00		$ 93,750.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	-	EA	$ 172,178.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 258,267.00		$ 258,267.00	Assume 1 per pipe penetration
	Activity SubTotal					$451,701	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Station						150 CFS pump station assumed at 2x cost of 50 CFS Example
	Screening Chamber	2.00	LS	$ 231,125.00		$ 462,250.00	
	Piping, Valves, Fittings	2.00	LS	$ 988,125.00		$ 1,976,250.00	
	Generator System	2.00	LS	$ 270,000.00		$ 540,000.00	
	Pump Station	2.00	LS	$ 1,482,750.00		$ 2,965,500.00	

	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	
	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	2.00	LS	$ 250,000.00		$ 500,000.00	
	Activity SubTotal					$7,815,215	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$11,774,274	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,766,141.09	
	Contractor Proffit			10%		$ 1,177,427.39	
	Engineering / Design			22%		$ 2,590,340.26	
	Contingency / Market Volatility			25%		$ 3,679,460.60	
	Total Including Contingencies					$20,987,643	

Study Area 5A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 131,042.89		$ 131,042.89	
	Maintenance of Traffic	1.00	LS	$ 56,161.24		$ 56,161.24	
	Temporary Erosion Control Measures	1.00	LS	$ 18,720.41		$ 18,720.41	
	Activity SubTotal					$205,925	
	Proposed Conveyance Improvements						
	Pavement Replacement	4,506.67	SY	$ 71.00		$ 319,973.33	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	20.00	EA	$ 1,875.00		$ 37,500.00	
	Remove & Dispose of Existing Pipes	-	LF	$ 55.00		$ -	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	-	CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves		EA	$ 37,625.00		$ -	
	Concrete curb and gutter	3,380.00	LF	$ 46.00		$ 155,480.00	
	Sidewalk Replacement	16,900.00	SF	$ 12.00		$ 202,800.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	20.00	EA	$ 8,100.00		$ 162,000.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	7.00	EA	$ 11,100.00		$ 77,700.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)		EA	$ 20,000.00		$ -	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	450.00	LF	$ 231.00		$ 103,950.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	1,130.00	LF	$ 390.00		$ 440,700.00	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	1,800.00	LF	$ 465.00		$ 837,000.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	3,380.00	LF	$ 31.25		$ 105,625.00	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	10.00	LS	$ 20,000.00		$ 200,000.00	Assume 1 per intersection
	Activity SubTotal					$2,749,809	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	5.00	EA	$ 1,875.00		$ 9,375.00	
	Grout and Seal Existing Well	5.00	EA	$ 80,000.00		$ 400,000.00	
	Activity SubTotal					$409,375	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 172,178.00		$ 172,178.00	Assume 1 per pipe penetration
	Activity SubTotal					$316,515	
	Proposed Road Elevation Adjustments						
	Black Base	565.50	Ton	$ 257.00		$ 145,333.50	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	10.00	EA	$ 2,425.00		$ 24,250.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	1,300.00	LF	$ 46.00		$ 59,800.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	3,250.00	SF	$ 12.00		$ 39,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$268,384	
	Proposed Stormwater Pump Station						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	

	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	
	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$3,950,007	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 592,501	
	Contractor Proffit			10%		$ 395,001	
	Engineering / Design			22%		$ 869,002	
	Contingency / Market Volatility			25%		$ 1,234,377	
	Total Including Contingencies					$7,040,888	

Study Area 5A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 336,192.29		$ 336,192.29	
	Maintenance of Traffic	1.00	LS	$ 144,082.41		$ 144,082.41	
	Temporary Erosion Control Measures	1.00	LS	$ 48,027.47		$ 48,027.47	
	Activity SubTotal					$528,302	
	Proposed Conveyance Improvements						
	Pavement Replacement	4,506.67	SY	$ 71.00		$ 319,973.33	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	20.00	EA	$ 1,875.00		$ 37,500.00	
	Remove & Dispose of Existing Pipes	-	LF	$ 55.00		$ -	Assume 1 CY per every 3.75 LF
	Abandon / Plug Existing Pipe	-	CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves		EA	$ 37,625.00		$ -	
	Concrete curb and gutter	3,380.00	LF	$ 46.00		$ 155,480.00	
	Sidewalk Replacement	16,900.00	SF	$ 12.00		$ 202,800.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement		SF	$ 12.00		$ -	Assume 6" thick concrete all areas (20'x12')
	Inlets / Manholes (12" - 24" pipe connections)	20.00	EA	$ 8,100.00		$ 162,000.00	Assume 1 per 250 LF of pipe, minimum and 2 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	7.00	EA	$ 11,100.00		$ 77,700.00	Assume 1 per 250 LF of pipe, minimum
	Inlets / Manholes (>48" pipe connections)		EA	$ 20,000.00		$ -	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe		LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	450.00	LF	$ 231.00		$ 103,950.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe		LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	30" Elliptical Stormwater Pipe		LF	$ 393.75		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	1,130.00	LF	$ 390.00		$ 440,700.00	Assume all pipe at 6'-8' depth
	36" Elliptical Stormwater Pipe		LF	$ 487.50		$ -	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	1,800.00	LF	$ 465.00		$ 837,000.00	Assume all pipe at 6'-8' depth
	48" Stormwater Pipe		LF	$ 540.00		$ -	Assume all pipe at 6'-8' depth
	48" Elliptical Stormwater Pipe		LF	$ 675.00		$ -	Assume all pipe at 6'-8' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe		LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth for pump station outfall

	Dewatering System Installation	3,380.00	LF	$ 31.25		$ 105,625.00	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	6.00	Months	$ 17,846.84		$ 107,081.03	1 month per pipe run - does not include additional measures at outfall
	Utility Conflict Allowance	10.00	LS	$ 20,000.00		$ 200,000.00	Assume 1 per intersection
	Activity SubTotal					$2,749,809	
	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	5.00	EA	$ 1,875.00		$ 9,375.00	
	Grout and Seal Existing Well	5.00	EA	$ 80,000.00		$ 400,000.00	
	Activity SubTotal					$409,375	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	1.00	EA	$ 172,178.00		$ 172,178.00	Assume 1 per pipe penetration
	Activity SubTotal					$316,515	
	Proposed Road Elevation Adjustments						
	Black Base	565.50	Ton	$ 257.00		$ 145,333.50	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	10.00	EA	$ 2,425.00		$ 24,250.00	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	1,300.00	LF	$ 46.00		$ 59,800.00	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	3,250.00	SF	$ 12.00		$ 39,000.00	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$268,384	
	Proposed Stormwater Pump Station						
	Screening Chamber	1.50	LS	$ 231,125.00		$ 346,687.50	
	Piping, Valves, Fittings	1.50	LS	$ 988,125.00		$ 1,482,187.50	
	Generator System	1.50	LS	$ 270,000.00		$ 405,000.00	
	Pump Station	1.50	LS	$ 1,482,750.00		$ 2,224,125.00	

	Electrical Platform & Stairs	1.50	LS	$ 149,000.00		$ 223,500.00	
	Electrical and Instrumentation and Control Work	1.50	LS	$ 489,982.50		$ 734,973.75	
	Landscaping / Screening Allowance	1.50	LS	$ 46,625.00		$ 69,937.50	
	Aesthetics Allowance	1.50	LS	$ 250,000.00		$ 375,000.00	
	Activity SubTotal					$5,861,411	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$10,133,796	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,520,069	
	Contractor Proffit			10%		$ 1,013,380	
	Engineering / Design			22%		$ 2,229,435	
	Contingency / Market Volatility			25%		$ 3,166,811	
	Total Including Contingencies					$18,063,492	

Study Area 6A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 85,823.85		$ 85,823.85	
	Maintenance of Traffic	1.00	LS	$ 36,781.65		$ 36,781.65	
	Temporary Erosion Control Measures	1.00	LS	$ 73,563.30		$ 73,563.30	
	Activity SubTotal					$196,169	
	Proposed Conveyance Improvements						
	Pavement Replacement	3,464.00	SY	$ 71.00		$ 245,944.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	-	EA	$ 1,875.00		$ -	
	Abandon / Plug Existing Pipe	-	CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	1.00	EA	$ 37,625.00		$ 37,625.00	
	Concrete curb and gutter	1,649.00	LF	$ 46.00		$ 75,854.00	
	Sidewalk Replacement	8,245.00	SF	$ 12.00		$ 98,940.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	648.00	SF	$ 12.00		$ 7,776.00	Assume 6" thick concrete all areas
	Inlets / Manholes (12" - 24" pipe connections)	2.00	EA	$ 8,100.00		$ 16,200.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	8.00	EA	$ 11,100.00		$ 88,800.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	470.00	LF	$ 231.00		$ 108,570.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	945.00	LF	$ 390.00		$ 368,550.00	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 8'-10' depth
	48" Stormwater Pipe	1,022.00	LF	$ 540.00		$ 551,880.00	Assume all pipe at 8'-10' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	-	LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth
	72" Stormwater Pipe	550.00	LF	$ 765.00		$ 420,750.00	Assume all pipe at 8'-10' depth
	Dewatering System Installation	2,987.00	LF	$ 31.25		$ 93,343.75	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	7.00	LS	$ 20,000.00		$ 140,000.00	Assume 1 per intersection
	Activity SubTotal					$2,307,773	

	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	-	EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well	-	EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	-	EA	$ 172,178.00		$ -	Assume 1 per pipe penetration
	Activity SubTotal					$144,337	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Stations						
	Screening Chamber	-	LS	$ 231,125.00		$ -	
	Site Work	-	LS	$ 839,500.00		$ -	
	Piping, Valves, Fittings	-	LS	$ 988,125.00		$ -	
	Generator System	-	LS	$ 270,000.00		$ -	
	Pump Station	-	LS	$ 1,482,750.00		$ -	
	Electrical Platform & Stairs	-	LS	$ 149,000.00		$ -	
	Electrical and Instrumentation and Control Work	-	LS	$ 489,982.50		$ -	
	Landscaping / Screening Allowance	-	LS	$ 46,625.00		$ -	

	Aesthetics Allowance	-	LS	$ 250,000.00		$ -	
	Activity SubTotal					$0	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$2,648,279	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 397,242	
	Contractor Proffit			10%		$ 264,828	
	Engineering / Design			22%		$ 582,621	
	Contingency / Market Volatility			25%		$ 827,587	
	Total Including Contingencies					$4,720,557	

Study Area 6A Engineer's Opinion of Probable Construction Cost

Line Item	Activity	Quantity	Unit	Unit Cost Low	Unit Cost High	Total Cost	Quantity Notes
	General Construction Measures						
	Mobilization	1.00	LS	$ 383,423.25		$ 383,423.25	
	Maintenance of Traffic	1.00	LS	$ 164,324.25		$ 164,324.25	
	Temporary Erosion Control Measures	1.00	LS	$ 328,648.50		$ 328,648.50	
	Activity SubTotal					$876,396	
	Proposed Conveyance Improvements						
	Pavement Replacement	3,464.00	SY	$ 71.00		$ 245,944.00	Quantity assumes 12' wide trench width over LF of pipe installation
	Demo existing manhole to 4' below grade and backfill	-	EA	$ 1,875.00		$ -	
	Abandon / Plug Existing Pipe	-	CY	$ 203.75		$ -	Assume 1 CY per every 34 LF for 12" pipe
	Flap Gate / Check Valves	1.00	EA	$ 37,625.00		$ 37,625.00	
	Concrete curb and gutter	1,649.00	LF	$ 46.00		$ 75,854.00	
	Sidewalk Replacement	8,245.00	SF	$ 12.00		$ 98,940.00	Assume 5' concrete wide sidewalk, 6" thick, all areas
	Driveway Replacement	648.00	SF	$ 12.00		$ 7,776.00	Assume 6" thick concrete all areas
	Inlets / Manholes (12" - 24" pipe connections)	2.00	EA	$ 8,100.00		$ 16,200.00	Assume 1 per 250 LF of pipe, minimum and 1 at each intersection corner
	Inlets / Manholes (>24" pipe connections)	8.00	EA	$ 11,100.00		$ 88,800.00	Assume 1 per 250 LF of pipe, minimum
	18" Stormwater Pipe	-	LF	$ 186.00		$ -	Assume all pipe at 6'-8' depth
	24" Stormwater Pipe	470.00	LF	$ 231.00		$ 108,570.00	Assume all pipe at 6'-8' depth
	30" Stormwater Pipe	-	LF	$ 315.00		$ -	Assume all pipe at 6'-8' depth
	36" Stormwater Pipe	945.00	LF	$ 390.00		$ 368,550.00	Assume all pipe at 6'-8' depth
	42" Stormwater Pipe	-	LF	$ 465.00		$ -	Assume all pipe at 8'-10' depth
	48" Stormwater Pipe	1,022.00	LF	$ 540.00		$ 551,880.00	Assume all pipe at 8'-10' depth
	54" Stormwater Pipe	-	LF	$ 615.00		$ -	Assume all pipe at 8'-10' depth
	60" Stormwater Pipe	-	LF	$ 690.00		$ -	Assume all pipe at 8'-10' depth
	72" Stormwater Pipe	550.00	LF	$ 765.00		$ 420,750.00	Assume all pipe at 8'-10' depth
	Dewatering System Installation	2,987.00	LF	$ 31.25		$ 93,343.75	Estimate LF of both minor and major pipe sizes
	Dewatering System Operation	3.00	Months	$ 17,846.84		$ 53,540.51	1 month per intersection - does not include additional measures at outfall
	Utility Conflict Allowance	7.00	LS	$ 20,000.00		$ 140,000.00	Assume 1 per intersection
	Activity SubTotal					$2,307,773	

	Plug Existing Gravity Wells						
	Demo Existing Well to 4' Below Pavement	-	EA	$ 1,875.00		$ -	
	Grout and Seal Existing Well	-	EA	$ 80,000.00		$ -	
	Activity SubTotal					$0	
	Outfall Improvements						
	Remove / Abandon Existing Outfall	-	CY	$ 203.75		$ -	Assume 1 CY per every 3.75 LF
	Temporary sheet pile for coastal work	2,250.00	SF	$ 28.44		$ 63,990.00	Assume a 50'x50' work area with 15' high sheets (embedded 2/3)
	Dewatering Measures at Outfall	1.00	Months	$ 17,846.84		$ 17,846.84	Assumes same features as typical system for pipes but with temporary sheet pile cofferdam in place - use 1 month
	Seawall Outfall Structure w/ check valve / flap gate < 36"	-	EA	$ 43,750.00		$ -	Assume 1 per pipe penetration
	Seawall Outfall Structure w/ check valve / flap gate > 36"	1.00	EA	$ 62,500.00		$ 62,500.00	Assume 1 per pipe penetration
	Three-Chamber Baffle Box < 36"	-	EA	$ 114,785.00		$ -	Assume 1 per pipe penetration
	Three-Chamber Baffle Box > 36"	-	EA	$ 172,178.00		$ -	Assume 1 per pipe penetration
	Activity SubTotal					$144,337	
	Proposed Road Elevation Adjustments						
	Black Base	-	Ton	$ 257.00		$ -	Using 145 pounds per cubic ft. (use SF x Thickness x 145)/2000
	Utility Valve / Manhole Adjustments	-	EA	$ 2,425.00		$ -	Assume 4 per intersection or 2 per 200 LF
	Curb and Gutter - independent of pipe runs	-	LF	$ 46.00		$ -	For locations where road is raised but stormwater pipe is not modified
	Sidewalk Replacement - independent of pipe runs	-	SF	$ 12.00		$ -	Assume 5' concrete wide sidewalk, 6" thick, all areas for locations where road is raised but stormwater pipe is not modified
	Driveway Replacement independent of pipe runs	-	SF	$ 12.00		$ -	Assume 6" thick concrete all areas for locations where road is raised but stormwater pipe is not modified
	Activity SubTotal					$0	
	Proposed Stormwater Pump Stations						
	Screening Chamber	2.00	LS	$ 231,125.00		$ 462,250.00	
	Site Work	2.00	LS	$ 839,500.00		$ 1,679,000.00	
	Piping, Valves, Fittings	2.00	LS	$ 988,125.00		$ 1,976,250.00	
	Generator System	2.00	LS	$ 270,000.00		$ 540,000.00	
	Pump Station	1.50	LS	$ 1,482,750.00		$ 2,224,125.00	
	Electrical Platform & Stairs	2.00	LS	$ 149,000.00		$ 298,000.00	
	Electrical and Instrumentation and Control Work	2.00	LS	$ 489,982.50		$ 979,965.00	

	Landscaping / Screening Allowance	2.00	LS	$ 46,625.00		$ 93,250.00	
	Aesthetics Allowance	1.00	LS	$ 250,000.00		$ 250,000.00	
	Activity SubTotal					$8,502,840	
	Proposed Pump Assisted Injection Wells						
	George Street Pump Station Example	-	LS	$ 1,822,230.00		$ -	
	Activity SubTotal					$0	
	Overall Subtotal					$11,831,346	
	Markups						
	Contractors Overhead, General Conditions, Temp Facilities			15%		$ 1,774,702	
	Contractor Proffit			10%		$ 1,183,135	
	Engineering / Design			22%		$ 2,602,896	
	Contingency / Market Volatility			25%		$ 3,697,296	
	Total Including Contingencies					$ 21,089,374	