Continuing Authorities Program (CAP)
Section 14 Project

Engineering Appendix
for
St. Francis Barracks Seawall
Emergency Shoreline Protection Project

Integrated Feasibility Report
and Environmental Assessment
St. Augustine, Florida

DRAFT

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of Engineers ®
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Contents

1 Background 1
1.1 Location 1
1.2 Problem Statement 2
1.2.1 Area of Concern 1 4
1.2.2 Area of Concern 2. 8
1.2.3 Area of Concern 3. 11
1.2.4 Area of Concern 4 13
1.2.5 Area of Concern 5 16
1.3 Past Studies 18
2 Alternatives 19
2.1 Initial Array of Management Measures 19
2.2 Evaluation Criteria 19
2.3 Alternative Formulation 19
2.4 Recommended Plan 20
3 Geotechnical Analysis 22
3.1 Regional Geology 22
3.2 Local Geology 24
3.3 Materials Encountered 25
3.4 Revetment Sizing 25
4 Coastal Engineering Analysis 26
4.1 4.1 Sea Level Change Analysis 27
4.2 Climate Change Analysis 28
4.2.1 Phase I: Relevant Current Climate and Climate Change. 29
4.2.2 Phase II: Projected Changes to Watershed Hydrology and Assessment of Vulnerability to Climate Change. 30
4.2.3 30
5 Pertinent Data 34
6 Civil Design 34

7 Real Estate 34
8 Relocations 35
9 Structural Requirements 36
10 Electrical and Mechanical Requirements 36
11 Hazardous, Toxic, and Radioactive Waste (HTRW) 36
12 Construction Procedures and Water Control Plan 37
13 Design and Implementation (D&I) Data and Tasks 37
14 Residual Engineering Risk 37
15 Operation and Maintenance 38
16 Schedule for Design and Construction 38
17 Resources 39

List of Figures

Figure 1-1 – Project Location 2
Figure 1-2 – Areas 1-4 of Concern 3
Figure 1-3 – Area 5 of Concern 3
Figure 1-4 – Southern Corner of Seawall 5
Figure 1-5 - Significant Cracking Near South Corner. 6
Figure 1-6 -Transition from Southern to Northern Wall Section. 7
Figure 1-7 – Void Space Beneath Patio Slab. 8
Figure 1-8 - Displaced Concrete Slab at JAG Building. 10
Figure 1-9 - Cracking of the JAG Building's Foundation. 11
Figure 1-10 - Gap Between Granite and Coquina Blocks. 12
Figure 1-11 – "Trench" Corresponding to the Gap shown in Figure 1-10. 13
Figure 1-12 – 12-inch Diameter Hole at Base of Wall. 14
Figure 1-13 - Detail of 12-inch Hole. 15
Figure 1-14 –Depressions in Ground Near 12-inch Hole in Seawall. 16
Figure 1-15 – Soil Within Water Flowing from Behind the Wall. 17
Figure 1-16 - Depression Corresponding to Area of Soil Loss. 18

Figure 2-1 – Selected Plan 22

Figure 3-1 - Present Coastline, Previous Sea Level Stands, and the Carbonate Platform of Florida Peninsula. 23

Figure 3-2. Map of Florida, project location and Atlantic Coastal Plain points of interest. 24

Figure 4-1 - NOAA Chart 11485. 26

Figure 4-2 – Sea Level Rise Curves 28

Figure 4-3 – Precipitation Changes 30

Figure 4-4 – Streamflow Data in St. Johns River 31

Figure 4-5 – Streamflow Data for Pellicer Creek 31

Figure 4-6- Observed Change in Very Heavy Precipitation 32

Figure 4-7 - Changes in Consecutive Dry Days 33

Figure 4-8 - Observed Increase in Frost-Free Season Length 33

Figure 6-1 - Access Roads, Staging and Storage Areas 35

Figure 6-2 – Storm Sewer Inlet 36

List of Tables

Table 1-1 – Wall Heights 4

Table 4-1 – Wave Information. 27

Table of Acronyms

ACES	Automated Coastal Engineering System
AOS	Apparent Opening Size
ASTM	American Society for Testing Materials
CAP	Continuing Authorities Program
CMU	Concrete Masonry Unit
CONUS	Continental United States
cy	Cubic yards
DI	Design and Implementation
ECB	Engineering and Construction Bulletin
EM	Engineer Manual
ER	Engineer Regulation
ESA	Environmental Site Assessment
ETL	Engineer Technical Letter
FANG	Florida Army National Guard
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIS	Flood Insurance Study
GPR	Ground penetrating radar
HUC	Hydrologic Unit Code
in	Inches
JAG	Judge Advocate General
lb	Pounds
lf	Linear feet
MLLW	Mean Lower Low Water
Mph	miles per hour
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
ROW	Right-of-Way
SLC	Sea Level Change
SLR	Sea Level Rise
SPT	Standard Penetration Test
USACE	U. S. Army Corps of Engineers
USGS	U. S. Geological Survey
VA	Vulnerability Assessment
VE	Value Engineering

Executive Summary

This appendix documents all engineering analyses and design tasks that were performed for the St. Francis Barracks Seawall Project (hereafter Project) Final Integrated Feasibility Report (Main Report). This Continuing Authorities Program (CAP) Project is authorized by Section 14 of the Emergency Streambank and Shoreline Protection, Flood Control Act of 1946, as amended. This engineering appendix includes alternatives evaluated, engineering analysis summaries, and the preliminary design. The engineering cost information is provided as a separate appendix. This document meets all requirements in Appendix C of ER 1110-2-1150.

1 Background

The study was requested by the Florida National Guard, the non-federal sponsor, in a letter dated October 21, 2016 (Pertinent Correspondence Appendix E). The intent of a CAP Section 14 Emergency Streambank and Shore Protection project is to stop or greatly reduce erosion within a short timeframe, by implementing a simple solution that requires minimal design and construction efforts.

1.1 Location

The project is located at the St. Francis Barracks, Florida National Guard Headquarters, St. Augustine, St. Johns County, Florida, as shown in Figure 1-1. This area contains visitors parking to the St. Francis Barracks Historical Site, military equipment for the Florida National Guard, Judge Advocate General (JAG) Office, parade ground, and the St. Francis Barracks themselves. The study area includes approximately 500 linear feet of an American-era, 19th Century seawall that runs along the shore of the Matanzas River along the St. Francis Barracks property. The coquina foundation was laid in 1842, topped with granite coping stones, and was completed in 1846. The seawall, parade ground, St. Francis Barracks and associated structures are part of the St. Francis Barracks Resource Group, which is eligible for listing on the National Register of Historic Places. The seawall, parade ground, and St. Francis Barracks are within the St. Augustine Historic District, which was listed on the National Register of Historic Places in 1971.

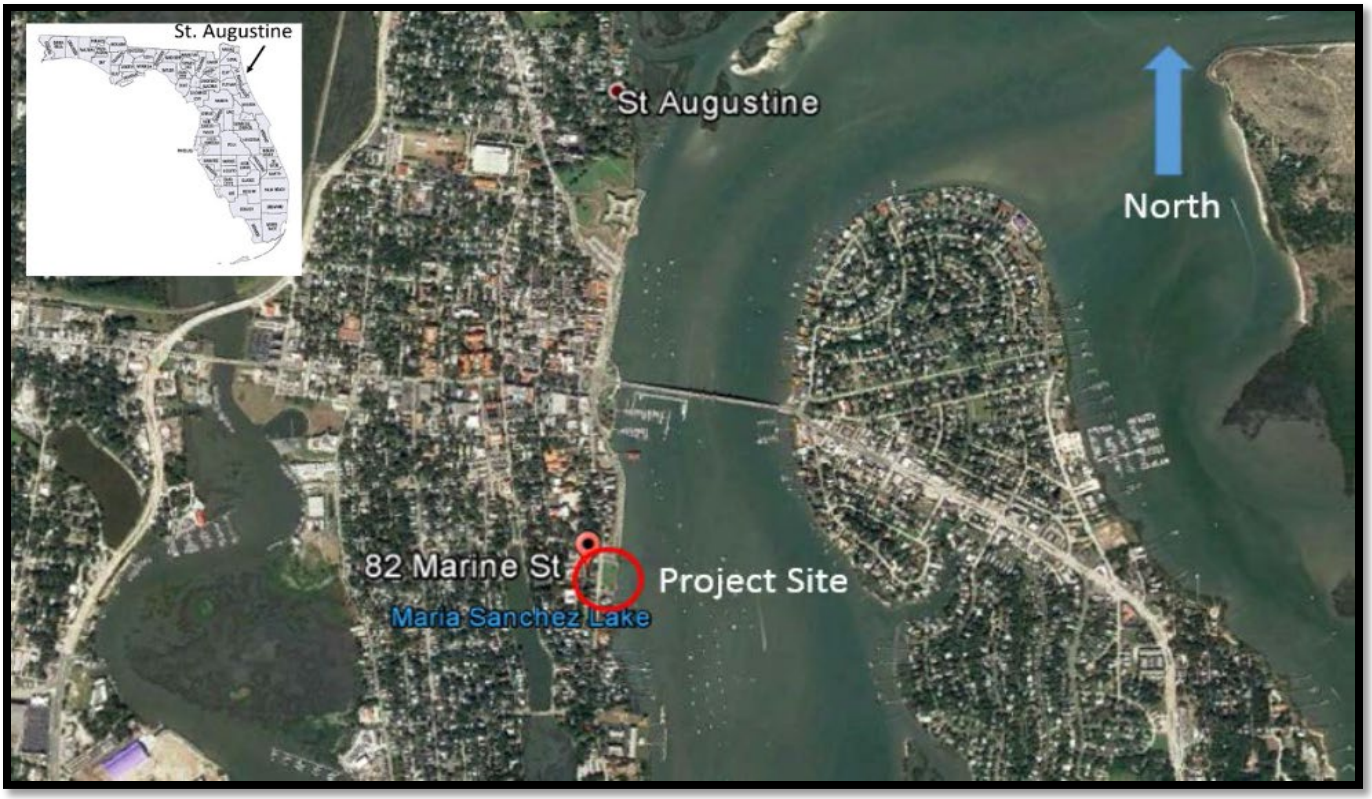

Figure 1-1 – Project Location

1.2 Problem Statement

The St. Francis Barracks section of the seawall has been overtopped many times in its history since construction was completed in 1846, generally during cases of extreme high tide combined with strong winds. When waves overtop the wall, the over-wash drains back out through cracks in the wall, taking soil with it. Maintenance operations by the Florida National Guard have included backfilling of depressions along the seawall with soil in the area of the parade ground every three to five years. The seawall was most recently overtopped during Hurricane Matthew (October 2016), resulting in undermining of the JAG office foundation, settlement, and cracking, as evidenced from the damage to the concrete pavement between the seawall and the JAG office building. The JAG office building was therefore evacuated and currently remains unoccupied. A site visit after Hurricane Matthew identified multiple depressions behind the seawall, where material had migrated through gaps in the seawall.

As stated above, the interpretation of this Authority is to stop the erosion with a simple solution that requires minimal design and construction. However, this project has a unique set of circumstances that requires careful consideration in determining the appropriate erosion protection design. Due to the age, unknown design standards, and historic significance of the project site, a minimally invasive design and construction methods are preferred.

The Project area is surrounded by medium density residential and commercial buildings on three sides, and

the Matanzas River on the east side. A site visit conducted in December 2016 found substantial erosion concerns along the wall which threaten infrastructure within the St. Francis Barracks Resource Group. Five areas of concern were noted in particular, shown in Figure 1-2 and Figure 1-3, and discussed in the following sections. During the initial site visit, the height of the waterside wall face was measured at four locations (from the top of the wall cap to the mudline/top of waterside ground surface). The yellow triangles in the figures show where wall height measurements were taken during this site visit. They are provided in Table 1-1.

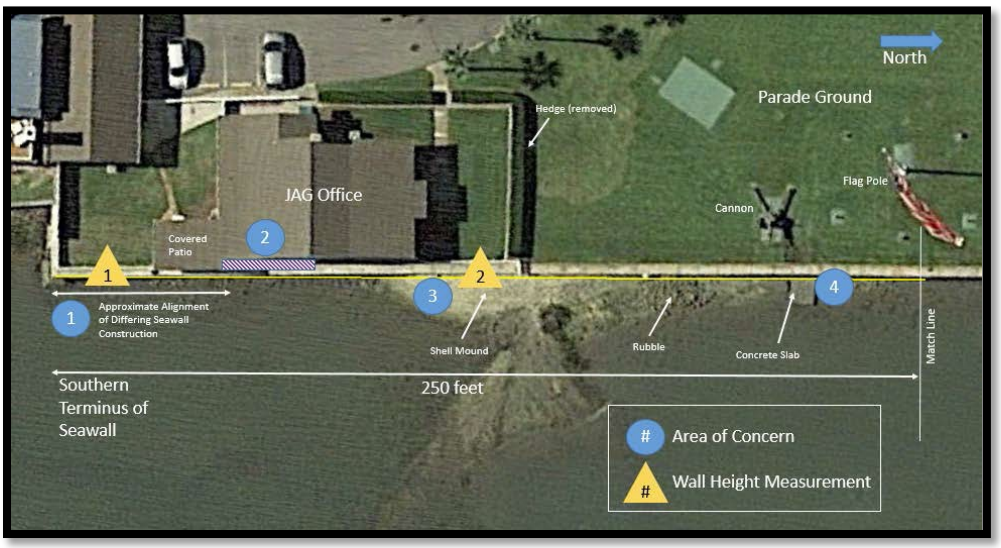

Figure 1-2 – Areas 1-4 of Concern

Figure 1-3 – Area 5 of Concern

Table 1-1 – Wall Heights

Measurement Location	Approximate Wall Height (feet, inches)	Ground surface along waterside face
1	5’ 6”	Mudline. Various rubble and debris.
2	3’ 0”	Shell mound.
3	6’ 0”	Mudline. Various rubble and debris.
4	2’ 6”	Sand/shell mound.

1.2.1 Area of Concern 1

The seawall in this area, beginning at the southern terminus and extending approximately 60 linear feet to the north, appeared to be constructed of different materials than the remainder of the wall to the north. Instead of the coquina and granite blocks, the wall in this area appeared to be constructed of bricks and concrete/mortar with concrete capping and Concrete Masonry Unit (CMU) curb. In addition to the general deterioration of the concrete veneer of the wall, significant cracking was also observed within the wall structure. Many of the CMU blocks of the curb had been lost, and appeared to be resting along the waterside base of the wall. A concrete buttress was observed at the southern terminus, where the wall alignment turns 90 degrees to the west, and appeared to have been constructed as an addition to this section. No active soil movement was observed through the wall in this area during the site visit, but the concrete slab of the covered patio along the south side of the JAG office building appeared to be generally undermined along this area of the wall. It appeared that there was approximately 6 to 8 inches of void space between the bottom of the slab and the underlying ground surface. The horizontal extents of the void space could not be measured. Given the extent of the subsidence below the slab, to the west of the wall, it is likely that the retained landside soil has been eroding through the cracks of the wall for some time. Figure 1-4 through Figure 1-7 show close-up views of this area of concern. Figure 1-4 shows the southern section of seawall looking northwest. The seawall is constructed of brick and concrete/mortar, with a concrete buttress at corner. Figure 1-5 is a detail of the southern wall section constructed of brick and concrete/mortar, and significant cracking in this section of wall. Figure 1-6 shows apparent transition from southern wall section (brick and concrete/mortar with concrete capping) to northern wall section (coquina block with granite coping blocks). Figure 1-7 shows the patio slab adjacent to seawall, and the visible void space along north edge of slab.

Figure 1-4 – Southern Corner of Seawall

Figure 1-5 - Significant Cracking Near South Corner.

Figure 1-6 -Transition from Southern to Northern Wall Section.

Figure 1-7 – Void Space Beneath Patio Slab.

1.2.2 Area of Concern 2.

A portion of the concrete slab, located between the landside side of the seawall and the east face of the JAG office building, approximately 18 inches wide and 30 feet long, showed signs of undermining, settlement, and cracking. The southernmost portion of this slab has experienced differential settlement, leading to rotation of the slab and exposure of the underlying soils. A void space was measured to be approximately 13 inches deep, below the bottom of the displaced slab portion. The exposed concrete portion of the office building's east

wall foundation showed significant cracks. Flowing water was observed exiting from the wall along the mudline (waterside). However, no soil particles were observed to be moved within this flowing water. The most severe area of slab damage appeared to coincide with the area where the wall transitions from brick and concrete/mortar into the coquina/granite block construction. Although landside soils appear to have been lost from behind the wall, the seawall itself appeared to be generally stable. It was reported that the damage to the slab and building foundation occurred as a result of Hurricane Matthew. However, it is more likely that the slab and foundation had been undermined for some time prior to the storm, due to soil loss (erosion) through the joints and cracks of the seawall structure as a result of flow of waters from high water events, tidal fluctuations, rain events, etc., and that the overtopping event caused by Matthew was just the catalyst that exacerbated the soil erosion and exposed the slab's weakened support condition, leading to slab displacement. Figure 1-8 shows the displaced portion of the concrete slab at the JAG building, landside of the seawall, and the cracks in the adjacent building foundation. Figure 1-9 shows cracking in a portion of the JAG office building's east wall foundation.

Figure 1-8 - Displaced Concrete Slab at JAG Building.

Figure 1-9 - Cracking of the JAG Building's Foundation.

1.2.3 Area of Concern 3.

A gap, approximately 2-inches high and 12 inches wide, was observed between the bottom of the granite coping stone and the coquina block. A "trench," approximately 8 inches wide, 12 inches deep, and 16 feet long was observed directly along the landside side of the wall in this area. The soils within this "trench" were visible from the waterside side when looking through the 2-inch gap noted above. Based on the appearance of new sod on the landside side of the wall in this area, and a comparison with Google Earth imagery of this site, it appears that landscaping work (removal of a hedge row) was performed within this area. Also, evidence of an irrigation line was observed along the immediate landside of the granite coping in the northern portion of the wall. It is unknown if this irrigation line previously extended into this area. It is unclear if the "trench" has any relation to the apparent landscaping work; however, the noted gap in the wall would certainly provide a conduit for erosion of soils from behind the wall during high water events, rainfall events, overtopping events, etc. A 4-inch diameter pipe (metal) was observed to exit the waterside face of the wall to the north of the gap area discussed above. No flow was observed exiting the pipe at the time of the site visit. Figure 1-10 shows a gap between the granite coping block and the coquina block, on the waterside wall face. The landside soils were visible through this gap. Figure 1-11 shows the "trench" on the landside side of the wall, corresponding to the gap area shown in Figure 1-10. Note newer sod in background of photograph (apparent area of former hedge row).

Figure 1-10 - Gap Between Granite and Coquina Blocks.

Figure 1-11 – "Trench" Corresponding to the Gap shown in Figure 1-10.

1.2.4 Area of Concern 4

A large hole, approximately 12 inches in diameter, was observed on the waterside face of the wall at the mudline, near the area between the southernmost cannon monument and the flagpole. The horizontal extent of the hole (into the wall) could not be determined. Depressions in the ground surface of the parade ground, approximately three to five feet in diameter and six to ten inches deep, were observed along the landside side of the wall, corresponding to the area of the hole noted above. Figure 1-12 shows the 12-inch diameter hole at the base of wall at the mudline. Figure 1-13 shows a detail of 12 inch hole in Figure 1-12. Figure 1-14 shows

the depressions in the landside ground surface in the area of the 12-inch hole in the seawall.

Figure 1-12 – 12-inch Diameter Hole at Base of Wall.

Figure 1-13 - Detail of 12-inch Hole.

Figure 1-14 –Depressions in Ground Near 12-inch Hole in Seawall.

1.2.5 Area of Concern 5

Flowing water was observed exiting from the wall along the mudline (waterside). Soil particles appeared to have been moved within this flowing water. A corresponding depression, approximately 3 feet in diameter and 4 to 5 inches deep, was observed at the ground surface on the landside side of the wall.

Figure 1-15 shows a detail of Figure 1-15, showing movement of soil particles within the water flowing from behind the wall. Figure 1-16 shows the depression on the landside side of the wall corresponding to the area of soil loss.

Figure 1-15 – Soil Within Water Flowing from Behind the Wall.

Figure 1-16 - Depression Corresponding to Area of Soil Loss.

1.3 Past Studies

In 2005, the City of St. Augustine contracted with Taylor Engineering to design and permit the construction of a new seawall on the waterside of the existing deteriorating seawall (CMAR 2015). The new Avenida Menendez Seawall, which was constructed approximately twelve feet east of the old seawall, began approximately 170 feet north of the Santa Maria Restaurant and extended south to the northern boundary of the FANG parking lot (Sastre 2005). The twelve feet of space between the old and new seawalls was filled with earthen fill to support a promenade for pedestrian traffic. The wall did not extend south in front of the

FANG parcel, as pedestrian traffic is not conducive to FANG's use of the grounds. Construction was completed in 2014. The Sponsor provided the Avenida Menendez Seawall Feasibility Study for this project (Sastre 2005), as well as the engineering plans (Taylor 2013), which were referenced for technical data.

2 Alternatives

2.1 Initial Array of Management Measures

The initial array of management measures included:

- Do nothing
- Construct a new wall
- Install anchor rods through wall
  - Vertically
  - Horizontally
- Seal cracks in the wall
  - Shotcrete
  - Flowable fill
  - Soil
  - Grout
- Sheetpile
  - Waterside
  - Landside
- Add weepholes
- Revetment
- Combination of the above alternatives

2.2 Evaluation Criteria

The following evaluation criteria was used to rate each management measure on how it meets the project objectives:

- Erosion Protection;
- Cultural Resources Impacts; and
- Relative Cost (level of construction scope).

Each Management measure was scored a 0, 1, or 2 for each criterion; 2 meaning it meets the objective.

2.3 Alternative Formulation

The management measures were combined in different permutations, resulting in ten unique management measure combinations, or alternatives. The sum of each alternatives' management measure scores was calculated for comparison. Due to the intent of CAP Section 14, hydraulic modeling, geotechnical and

structural analyses, and cost estimates for each alternative were not performed, but instead were replaced with a qualitative analysis of the alternatives. The alternative with the highest score was carried forward. A detailed discussion of the individual alternatives and evaluation process is provided in the Main Report.

2.4 Recommended Plan

As a result of the qualitative analysis presented in Section 2.3, the most advantageous alternative was determined to be a combination of the following management measures:

- Install new weep holes where necessary;
- Fill wall voids with grout (waterside);
- Place flowable fill (or an equivalent granular, free-draining material) (landside);
- Backfill topsoil;
- Apply shotcrete at south corner (waterside); and
- Place revetment at south corner (waterside).

Weep holes are critical for providing groundwater discharge points to alleviate hydrostatic pressure. If excess groundwater (from rainfall and wave overwash) is not allowed to freely discharge, the structural loading on the landside of the wall will increase, possibly resulting in damage to the structural integrity of the wall. The design includes weep holes to be installed approximately every 10'. The spacing, landside extension, and bedding material of weep holes will be verified after the seepage analysis to be performed during the Design and Implementation phase (DI). Each weep hole will include gravel drainage.

Grout will be used to fill the large cracks in the wall on the waterside in order to prevent the flowable fill (or an equivalent granular, free-draining material) from discharging into the Matanzas River. Flowable fill (or an equivalent granular, free-draining material) will be placed at the voids from the ground surface, and allowed to flow into the voids beneath the surface. The broken concrete sidewalk extending between the JAG building to the seawall will be removed for easier access to the voids beneath the building. In order to ensure the existing soil loading conditions on the landside of the wall are maintained, only minimal localized excavation of soil from the landside of the wall is planned to gain better access to the voids. Excavation will occur only locations known to have voids, and will be done only at the surface mainly by hand (or mini excavator if necessary). At present, the "known" voids are the five discussed in Section 1, based upon visual inspection. During DI, the subsurface investigation may reveal that more voids exist and need filling. Flowable fill (or an equivalent granular, free-draining material) will be placed at the five main areas of erosion concern where large voids exist as shown in Figure 2-1. The plan reduces the need to excavate behind the wall by limiting flowable fill (or an equivalent granular, free-draining material) placement into the five main areas of concern that are clearly visible and have high surface settlement. Flowable fill (or an equivalent granular, free-draining material) will not only block the preferential pathways of drainage to the large openings, but it will also prevent further settling of the wall (which increases the opening sizes) by preventing further soil from eroding, especially at the south corner. Construction methods will be used to ensure spreading of flowable fill (or an equivalent granular, free-draining material) is not inhibited by trapped air, and will be used to limit flowable

fill (or an equivalent granular, free-draining material) only spreads where it is needed and does not clog the drainage system (such as vent pipes and backer rods). Topsoil and sod will be placed on top of flowable fill (or an equivalent granular, free-draining material) in order to maintain the original appearance, and the concrete sidewalk in front of the JAG building will be replaced. During the DI phase, the appropriate grout and flowable fill (or an equivalent granular, free-draining material) mixes will be determined. The design does not include performing any repairs to the JAG building foundation or piping. The seepage analysis performed during DI to refine the weep hole spacing may also reveal that the flowable fill mixture may need to be replaced with a granular, free-draining material that meets filter criteria for the soil retained behind the wall. Filter criteria for the soil behind the wall will be determined with the subsurface analysis to also be performed during DI.

Shotcrete with fiber reenforcement will be applied on the waterside of the south corner in order to add additional erosion control. In order to ensure that the shotcrete remains in place, an anchoring frame will be connected to the wall. From the corner, the shotcrete will extend north 40 linear feet (lf), and west approximately 20 lf to one foot east of the next corner. During the Value Engineering (VE) Study in DI phase, it may be determined that a similar substance will result in the same desired outcome with a more cost-effective installation process, such as quick set mortar or cement.

Lastly, stone revetment and bedding stone will be placed at the corner in order to provide a wave break to reduce any wave-induced erosion to the structure. The stone revetment will have a maximum radius of 25 feet from the corner to the north. The radius of the revetment tapers off to about 20 feet moving west, to within one foot of the adjacent property to the south. Based on the wave climate in this area, the appropriate stone size diameter was approximated at 2.5-ft, which was sized to handle waves generated by 3-hr sustained winds of between 115 and 125 mph. However, the schedule and budget did not allow for a detailed revetment analysis during the feasibility study. Therefore, additional revetment analysis will be conducted during the DI to verify the appropriate revetment design. The design includes only one layer of stone against the wall. To maintain a uniform stone elevation, an additional layer of stone may be necessary towards the outside of the radius of coverage as the Matanzas River invert slopes down.

It is important to note that the Recommended Plan addresses the current soil-loss conditions caused by runoff and wave overwash drainage through large cracks in the seawall under current conditions. The current conditions analyzed include storm conditions for the 100-year storm surge with sustained 150-mph winds. It is not designed to prevent wave overwash, or further damage to the waterside wall face by extreme wave action during high storm/Hurricane events. Residual risk remains that extreme high storm events in the future could excessively overtop and impact the seawall, causing further erosion thus damaging the facilities and ultimately impacting the structural stability of the seawall or Historic JAG building, which the Recommended Plan is not designed to address.

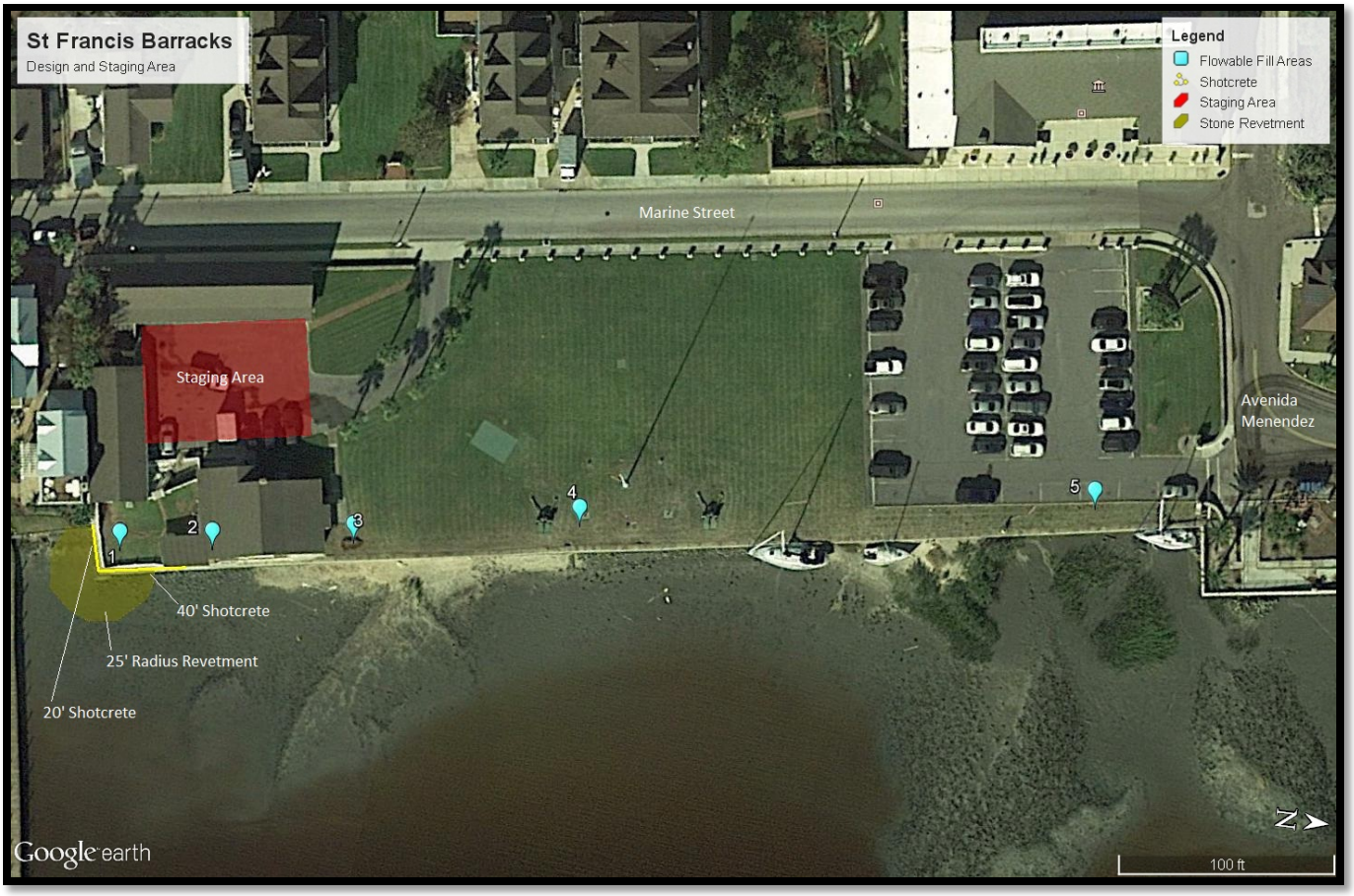

Figure 2-1 – Selected Plan

3 Geotechnical Analysis

3.1 Regional Geology

The Florida Peninsula occupies a portion of a much larger geologic unit called the Florida Plateau. Deep water in the Gulf of Mexico is separated from deep water of the Atlantic Ocean by this partially submerged platform, nearly 500 miles long, that varies from 250 to 450 miles wide. In the last 200 million years, the plateau has been alternately dry land or covered by shallow seas. During that time up to 20,000 feet of carbonate and marine sediments were deposited. There has been a tilting of the Florida Plateau about its longitudinal axis. The west coast is partially submerged, as indicated by the wide estuaries and offshore channels, while the east coast is correspondingly elevated, showing the characteristics of an emergent coastline (Randazzo and Jones, 1997).

During the last million years, a series of four glacial periods, or ice ages, brought about significant changes in sea level, as shown in Figure 3-1**.** As a result of these sea level fluctuations, the Florida peninsula was again

covered and uncovered by shallow seas. Following the first glacial period, sea level rose 270 feet above its present level. Dry land on the Florida peninsula was then restricted to a few small islands along the central Florida ridge and in northeast Florida.

About 100,000 years ago, the last glacial period began. Sea level fell to 300 feet below its present level and the Florida Plateau emerged as dry land. Approximately 15,000 years ago, sea level began its most recent rise towards present sea level (Shackleton, 1987). Sea level rose at an average rate of 30 feet per 1,000 years. About 7,000 years ago, the rate of sea level rise slowed when the sea level was about 30 feet below its present level (Smith et al., 2011). It was during this most recent slowing of sea level rise that the modern barrier islands of southeast peninsular Florida formed.

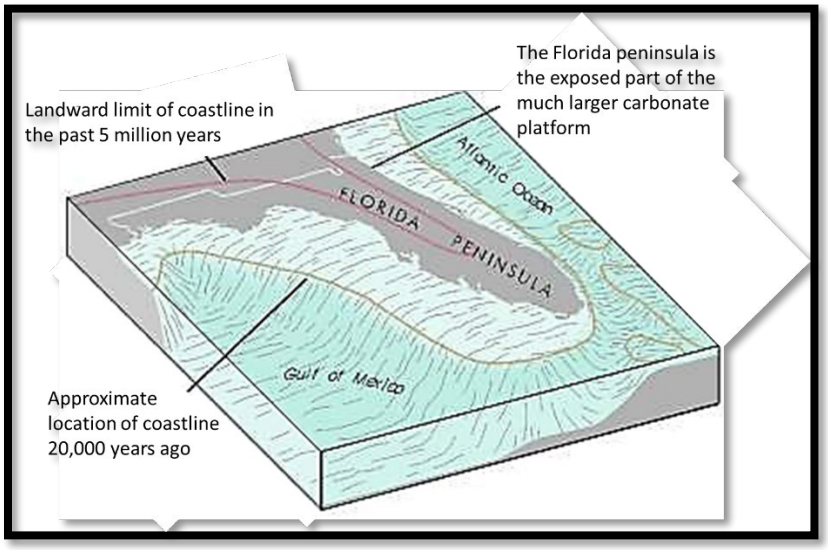

Figure 3-1 - Present Coastline, Previous Sea Level Stands, and the Carbonate Platform of Florida Peninsula.

Offshore of the beaches and modern barrier islands is the continental shelf. It is a broad, shallow, low relief shelf that extends from 80 miles offshore near Jacksonville, to only a few miles offshore near Miami. The shelf contains relic Pleistocene and Holocene terraces and submerged beach sand ridges. The wave climate and sediment transportation system creates a linear sandy coastline. The linear coastline is modified locally by inlets. An exception to the linear coastline is the cape structure located at Cape Canaveral which formed in response to a different wave and sand sediment transport system in the southern portion of the state.

The east coast of Florida, from the state line at the Georgia border to Miami Beach (350 miles), consists of a series of sandy barrier islands broken occasionally by inlets, as shown in Figure 3-2. The barrier islands are characterized by dunes and shore parallel beach ridges. Many of the islands display relic beach ridges formed during higher stands of sea level. The barrier islands often have a distinctive drumstick-shape with an accreting bulbous end and a slender eroding end. These barrier islands were formed from waves and longshore currents reworking marine and fluvial sediments. Lagoons and marshes are typically located between the barrier islands and the mainland.

The quartz component of the modern barrier island sand has deposited from sand migrating southward along the Atlantic coast, from the reworking of the Pamlico Sand that was previously deposited over the entire region. The remaining component of coastal sediments are typically carbonates, locally produced by calcite-producing plants and animals. Additional carbonate materials are from reworked materials from outcropping Pleistocene formations offshore (Duane and Meisburger, 1969).

Figure 3-2. Map of Florida, project location and Atlantic Coastal Plain points of interest.

3.2 Local Geology

The local geology of St. Johns County for the Quaternary and upper Tertiary Systems range in age from Recent to Pleistocene to Miocene age sediments. The formations exposed at the surface are undifferentiated sediments and the Anastasia Formation of Pleistocene and Recent age (Scott, et al., 2001). These deposits

consist of fine to medium quartz sand and lenses of shell and clay of varying thickness. Thick shell beds and erosion of the outcropping Anastasia formation near the coast have been firmly cemented to form coquina. This formation is underlain by Upper Miocene or Pliocene deposits of interbedded lenses of marine, fine to medium sand, shell and green, calcareous, silty clay. This is underlain by the Hawthorne Formation of early and middle Miocene age, the surface of which is approximately 130 feet below sea level. The Hawthorne Formation consists of gray to green, plastic, phosphatic, sandy clay and marl, interbedded with lenses of phosphatic sand, pebbles and sandy limestone. The Hawthorne Formation is underlain by limestone formations of Eocene age.

The project is located on the barrier beach on the Atlantic Ocean, in central St. Johns County, in the Atlantic Coastal Plain physiographic unit. The St. Johns County shore is a barrier beach with a low tidal marsh and lagoon behind it. For the northern 6 miles the beach ridge is about 3 miles wide, with dune elevations ranging from 15 to 25 feet, mean low water. For the next 12 miles the ocean is separated from the mainland by two ridges and two low marshes. The easterly ridge is about 500 to 1500 feet wide, with a nearly continuous dune line ranging in elevation from 15 to 44 feet. The eastern marsh, which contains the Guana River, is generally 2,000 feet wide. The land ridge west of the Guana River is generally 10 feet high and about 4,000 feet wide. The main marsh ranges in width from 3,000 feet to 9,000 feet wide and contains the Tolomato River at a point about 18 miles south of the Duval-St Johns County line.

The St. Johns County barrier islands have inlets at St. Augustine and at Fort Matanzas. There are low tidal marshes and lagoons between the barrier islands and the mainland. The barrier Islands are composed principally of sand and are underlain by silty, clayey marsh deposits that had formed at lower sea level stages.

3.3 Materials Encountered

Three (3) borings were provided by the Florida Geological Survey within the vicinity of the project site. The materials encountered include fine to coarse-grained sand-sized quartz with variable amounts of shell and shell fragments (sand to cobble size), silt, and clay. Rock is present at a depth of over 200 feet below ground surface, and will not be encountered during the project.

3.4 Revetment Sizing

The revetment was sized using the following combination of engineering methods and data:

- Revetment stone sizing formulas presented in Chapter 2 of Engineering Manual (EM) 1110-2-1614, *Design of Coastal Revetments, Seawalls, and Bulkheads (USACE 1995);*
- Revetment material determination per EM, Appendices B-E (USACE 1995);
- Revetment stone sizing formulas (1) and (2) presented in Coastal Engineering Technical Note (CETN) III-37, *Sizing of Toe Berm Armor Stone on Rubble-Mound Breakwater and Jetty Trunks Designed for Depth-Limited Breaking Waves* (USACE-WES 1987); and
- Wave data provided by the team Coastal Engineer, which is discussed in Section 4.

Armor unit stability equation (Eqn 2-15, (USACE 1995)) was used to determine the median stone weight based upon assuming breaking waves of the maximum wave height provided. This condition would occur during the sustained wind conditions (discussed in Section 4) and low tide.

4 Coastal Engineering Analysis

St. Francis Barracks is located on the western shore of the Matanzas River just south of the St. Augustine City Dock and the Bridge of Lions. Water depths immediately adjacent to the site are very shallow but further offshore and along much of the fetch length to the northeast the Matanzas River runs with depths between 20 and 30-ft mean lower low water (MLLW). The current Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) (12109CV000C) depicts a 100-year flood elevation of approximately 7.1-ft NAVD88. This equates to approximately 10 feet above MLLW. The National Oceanic and Atmospheric Administration (NOAA) Chart 11485 in Figure 4-1 shows the project location and the approximate 2.25-mile fetch length to the northeast. The top of wall elevation is approximately 5.840-ft NAVD88 (Taylor 2013).

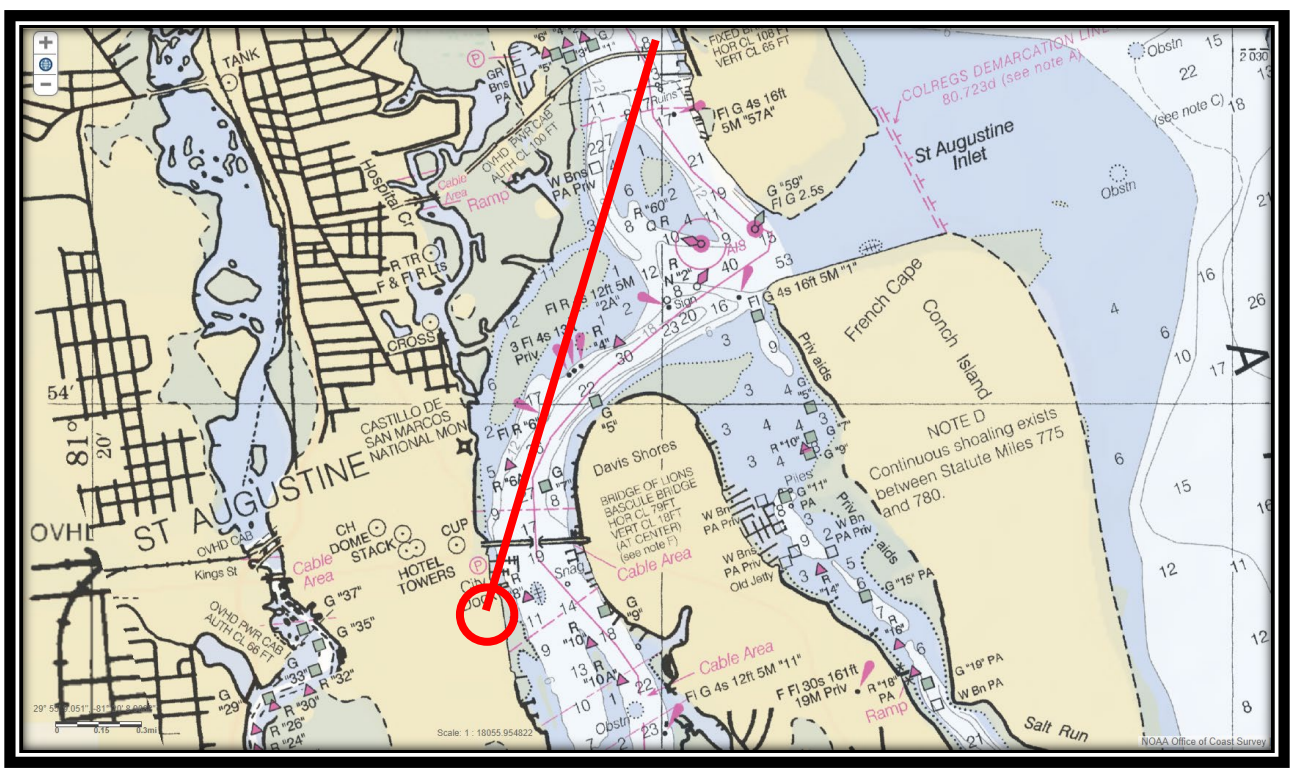

Figure 4-1 - NOAA Chart 11485.

The USACE program ACES (which stands for Automated Coastal Engineering System) was used to generate wave growth based on the 2.25-mile fetch and an estimated average water depth of 20 feet (including surge) across that fetch for different wind speeds assuming a three hour duration of the observed wind (typical hurricane conditions). Table 4-1 depicts the outcome of these estimates. Wave size is not only limited by the

fetch length but also by water depth which induces breaking. Typically waves break when the wave height reaches about 0.78 of the water depth. With essentially zero water depth fronting the barracks at mean low water, wave impacts will be limited to the maximum wave height allowed for the given surge elevation, or approximately a 7.8-ft wave height (assuming 10 feet of surge). Based on the ACES calculations this wave height would occur with wind speeds between 115 and 125 mph (between scenarios 5 and 6 in Table 4-1).

Table 4-1 – Wave Information.

Scenario	Average Water Depth (ft)	Fetch Length (miles)	Wind Speed (mph)	Peak Wind Duration (hours)	Wave Height (ft)	Wave Period (s)	Wave Length (ft)
1	20	2.25	75	3	4.44	3.31	48
2	20	2.25	85	3	5.15	3.5	52
3	20	2.25	95	3	5.87	3.68	56
4	20	2.25	105	3	6.59	3.85	59
5	20	2.25	115	3	7.31	4.01	62
6	20	2.25	125	3	8.03	4.16	65

4.1 4.1 Sea Level Change Analysis

ER 1100-2-8162, *Incorporating Sea Level Change (SLC) in Civil Works Programs*, provides regulations and guidance for incorporating direct and indirect physical effects of projected future sea level change to USACE Civil Works projects. These regulations apply to all USACE Civil Works activities and projects, both existing and proposed, across the project life cycle in managing, planning, engineering, designing, constructing, operating, and maintaining USACE projects and systems of projects. Consideration of potential relative sea level change is required in every USACE coastal activity as far inland as the estimated tidal influence, including studies that calculate backwater profiling with the ocean as the downstream boundary condition. Designs should be formulated and evaluated for three possible rates of sea level change; low, intermediate, and high SLC, which can be obtained for NOAA tide gauges at https://www.usace.army.mil/corpsclimate. The relative SLC curves should be applied to both with and with-out project conditions. Once the three levels of SLC are defined for the project location, analyses shall be performed to determine sensitivity and adaptability of the natural and managed ecosystems, as well as human and engineered systems to climate change.

NOAA's Published low (or historic) rate of SLC for Mayport (Bar Pilots Dock), FL, Gauge # 8720218, is +0.04 feet per year, which is the rate used to calculate the estimated future low amount of SLC. The Corps Climate SLC curve calculator, found at http://corpsmapu.usace.army.mil/rccinfo/slc/slcc_calc.htm, was used to produce Figure 4-2**,** which shows the estimated relative sea level changes for all three rates from 2019 to 2100, based on NOAA's published rates. The curve calculator automatically generates curves up to the year 2120 starting with a user-determined project start date. The graph shows the relative increase in sea levels based on the three USACE curves.

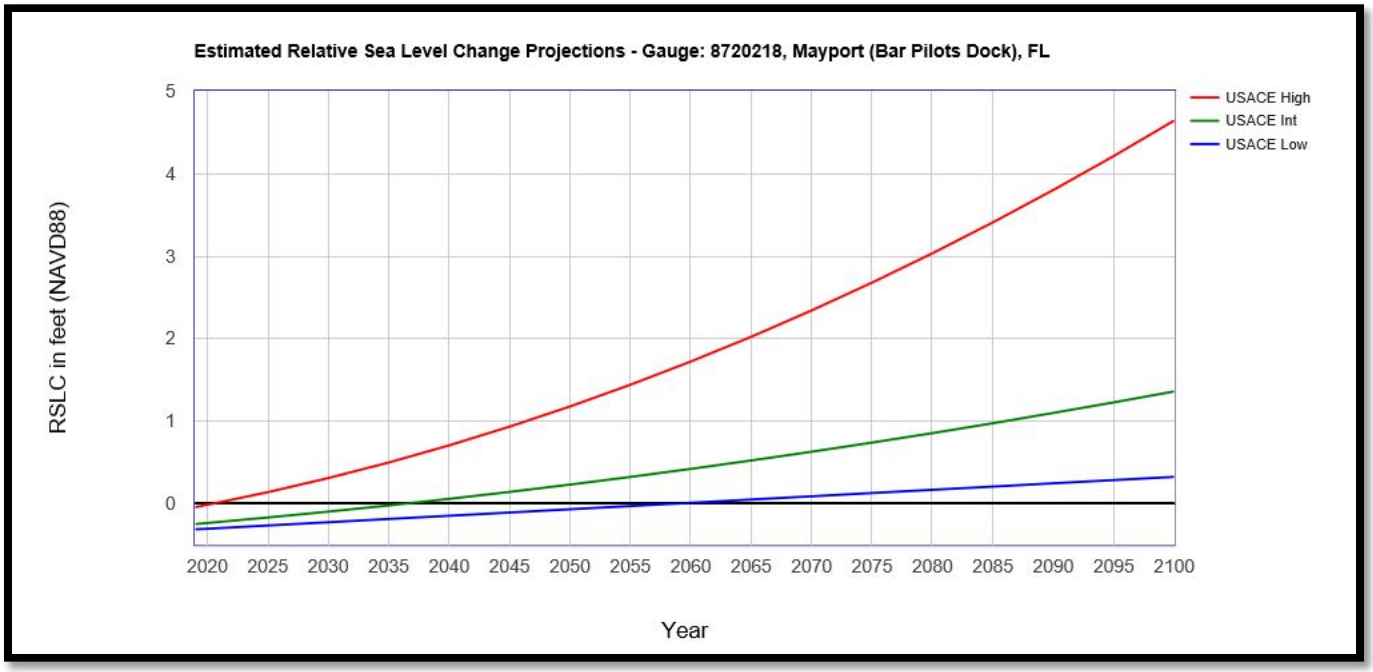

Figure 4-2 – Sea Level Rise Curves

The design includes reducing soil loss through existing cracks in the wall. However, it is important to note that at the high rate, the project site will be below sea level during high tide in 2100. At that time, the scope of this erosion protection design will not be applicable, and it is recommended that the wall height be raised by that time.

4.2 Climate Change Analysis

The overarching USACE climate change policy document, *USACE Climate Preparedness and Resilience Policy Statement* (June 2014), requires consideration of climate change at every step in the project life cycle for all existing and planned USACE projects to reduce vulnerabilities and enhance the resilience of our water-resource infrastructure. Guidance for incorporating climate change and hydrologic analyses is provided in Engineering And Construction Bulletin (ECB) No. 2016-25 (16 Sept 2016), *Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects*. This applies to all current and future studies and any completed projects for which Federal funds are being used to rehabilitate a project, but does not apply to short-term water management decisions. The analysis provides for consideration of specific climate change projections in the project area and potential impacts to the particular hydrologic analysis.

The required qualitative analysis involves two phases. Current climate change trends are analyzed during Phase I, and projected future changes to hydrology are analyzed during Phase II. Phase I consists of literature review and investigation of annual maximum stream flow trends using the USACE Climate Hydrology Assessment and USACE Nonstationarity Detection Tools. Phase II consists of investigating projected future

trends in annual maximum stream flows using the same two USACE tools mentioned previously, and performing a vulnerability assessment using the USACE Watershed Vulnerability Assessment Tool. The Climate Change assessment for this project is presented in the following sections. Given a limited budget and schedule, not all sections of the required Climate Change analysis could be completed during the feasibility study, and will be completed during DI.

4.2.1 Phase I: Relevant Current Climate and Climate Change.

4.2.1.1 Current Climate in St. Augustine, Florida.

St. Augustine, Florida, has a warm climate characterized by relatively high temperatures and humidity. The warmest month is July, with an average maximum temperature of 88.8°F; and the coolest month is January, with an average minimum temperature of approximately 42°F. The annual average rainfall is 39.37 inches. August is the wettest month with an average monthly precipitation of over eight inches, and November is the driest month with an average monthly precipitation of approximately two inches (https://weather-and-climate.com). The Hydrologic Unit Code (HUC) for this watershed is 030801 (https://water.usgs.gov/GIS/huc_name.html#Region03).

4.2.1.2 Observed Changes.

The annual average temperature in the Southeast United States has increased by 0.46°F since the beginning of the 20th century (Vose et al 2017). Over the period of 1901-2015 studied for the *Precipitation Change in the United States* report, annual precipitation changes vary across the United States, as well as seasonally. The project area experienced an 0-5% annual precipitation decrease (Easterling et al 2017). However, when analyzed seasonally, precipitation increased during the winter and summer seasons. The trend of increased rainfall volume during larger storms is discussed in Section 4.2.2. The annual and seasonal precipitation change results map for the United States is presented in Figure 4-3. Additional historic data was obtained from The NOAA National Environmental Satellite, Data and Information Service (NESDIS) report assessing climate trends and scenarios into the next 50–100 years for the Southeast Continental United States (CONUS) region (NOAA 2013). The report indicates that over the period of hydroclimatological record for the Southeastern United States, both temperature and precipitation have shown either a statistically insignificant trend or no trend in change historically.

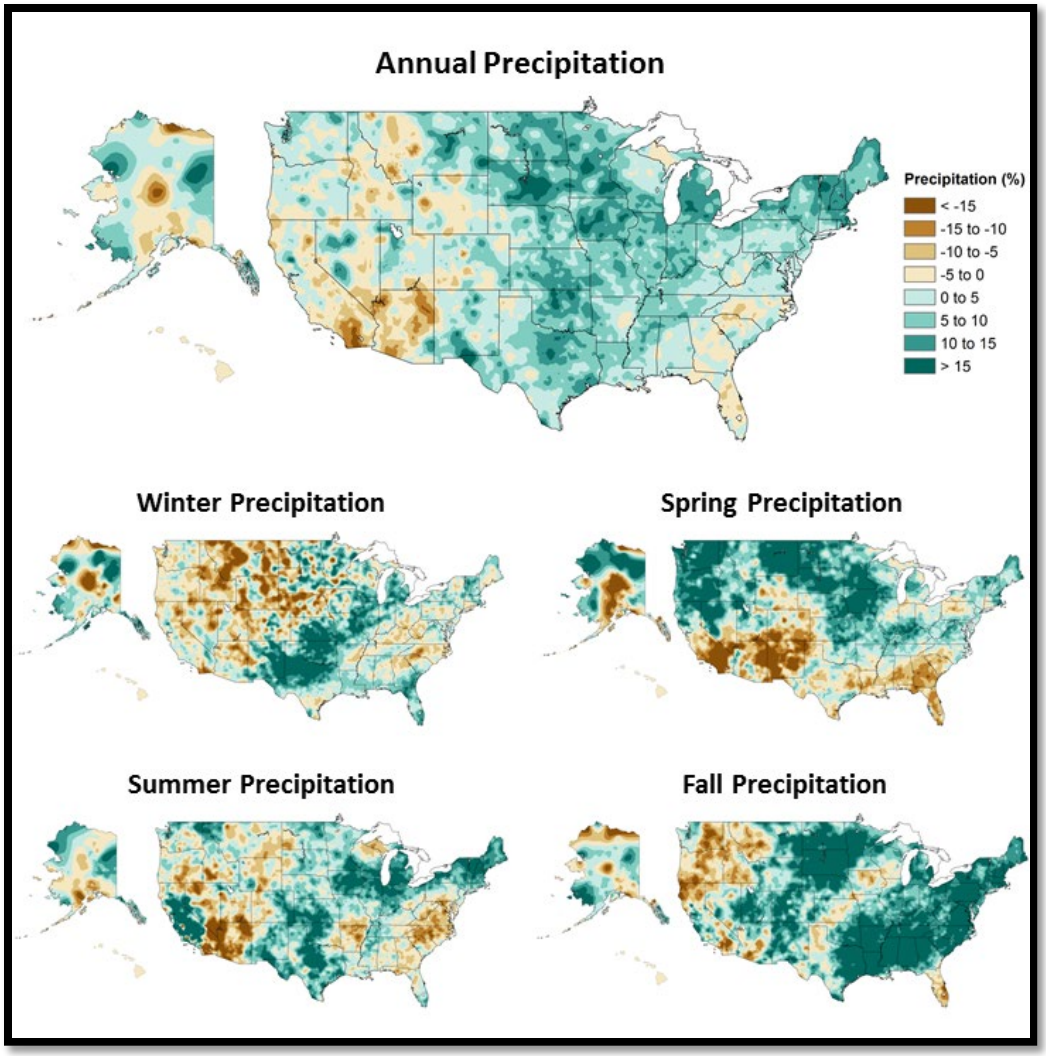

Figure 4-3 – Precipitation Changes

4.2.2 Phase II: Projected Changes to Watershed Hydrology and Assessment of Vulnerability to Climate Change.

The 2013 NOAA NESDIS report projected meteorological conditions in the region through 2099. According to the NESDIS report, a warming trend of approximately 2-5°F and no discernable precipitation trend can be expected over the next 50 years, although these estimates have significant uncertainty.

4.2.3

The USACE Climate Hydrology Assessment Tool shows annual streamflow values for given site locations. The project location is located between two streamflow gauge sites; one in the St. Johns River to the north, and one in Pellicer Creek to the south. The next two figures show the Annual Peak Instantaneous Streamflows over time for both sites. The St. Johns River site shows an increasing trend, while the Pellicer Creek site shows a decreasing trend in annual peak streamflows. Given this information, no assumptions can be made for streamflow trends in the Matanzas River.

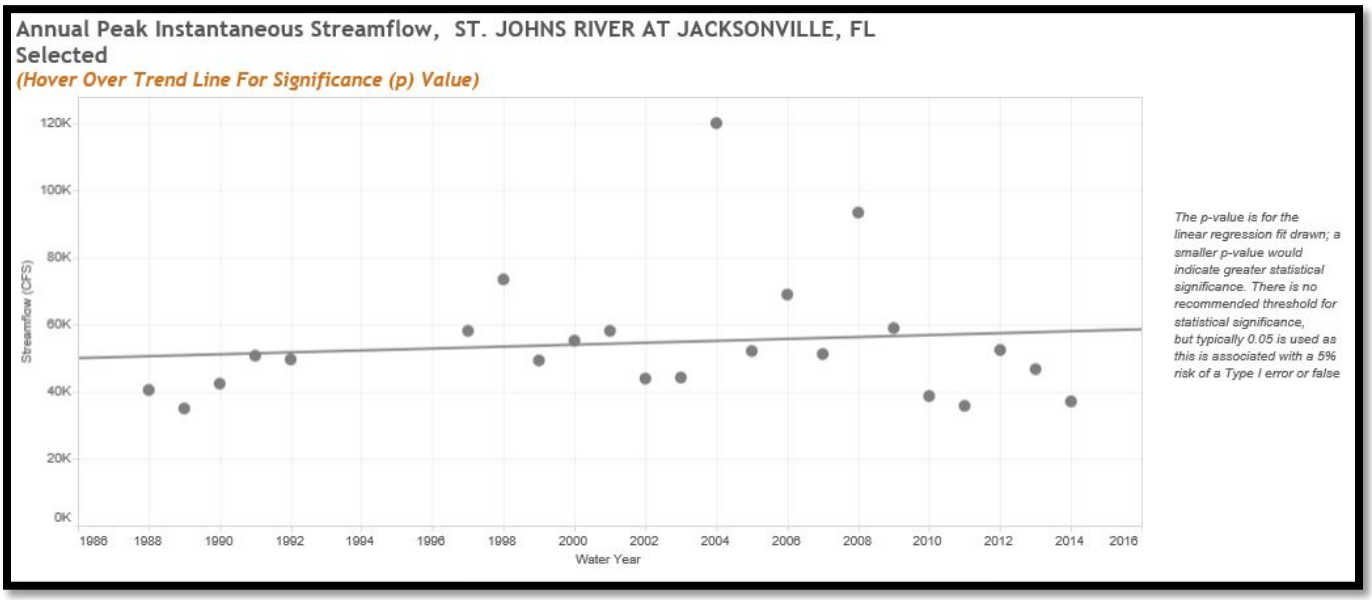

Figure 4-4 – Streamflow Data in St. Johns River

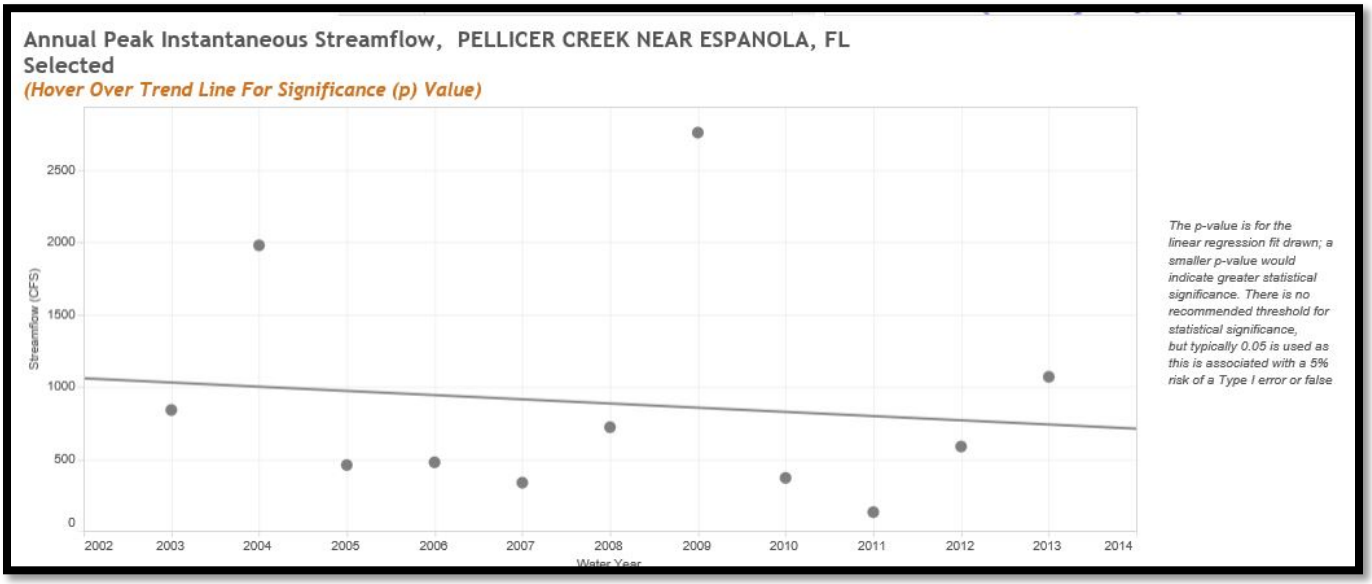

Figure 4-5 – Streamflow Data for Pellicer Creek

The USACE Watershed Vulnerability Assessment (VA) Tool contained data on precipitation and temperature trends in the Southeastern United States. The Regional Overview for the Southeast United States discusses threats to three key topics; increased sea level rise threats, increasing temperatures, and decreased water availability. For specific precipitation trends, this tool shows that Florida has experienced a 27% increase between 1958 and 2012 in precipitation amount during very heavy rain events (Figure 4-6). The tool also reports a modeled prediction of an over 10% increase in consecutive dry days in southeast for the years 2070-

2099 (as compared to the years between 1971 and 2000), if continued emissions increase (Figure 4-7). Regarding temperature trends, the VA Tool shows an average increase in the annual number of frost-free days between 5-9 days in the Southeastern US (Figure 4-8). The increased number of consecutive dry days combined with the higher temperatures and increased severity in large rainfall events has significant implications for native Florida flora and fauna, increased soil erosion, and human health.

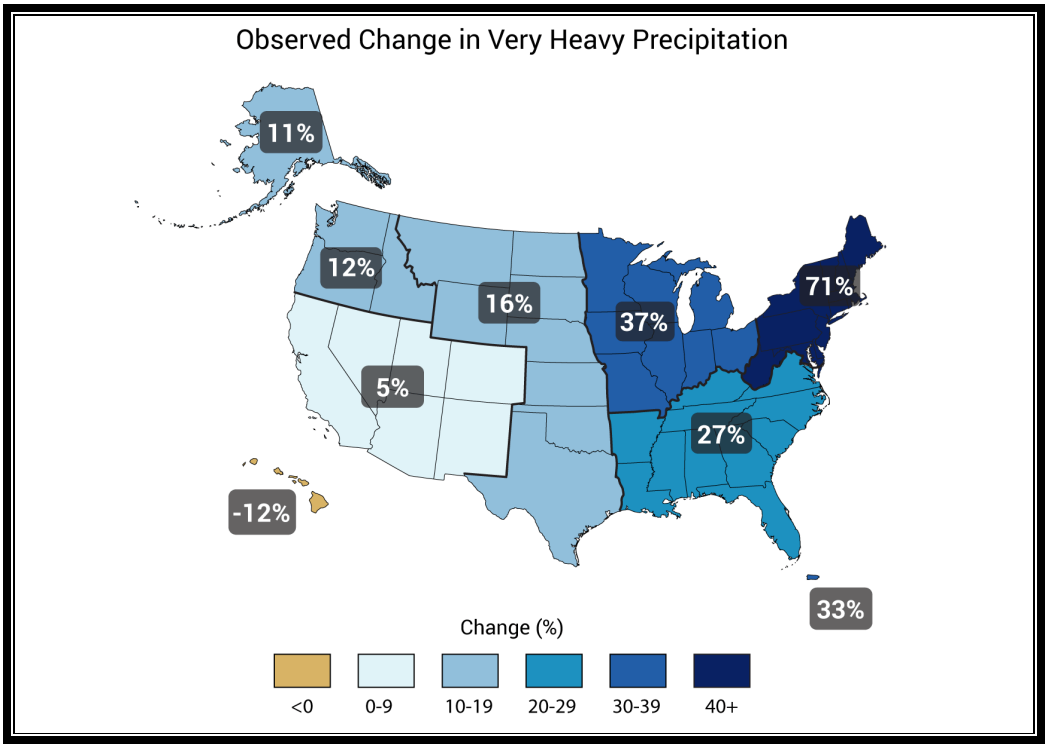

Figure 4-6- Observed Change in Very Heavy Precipitation

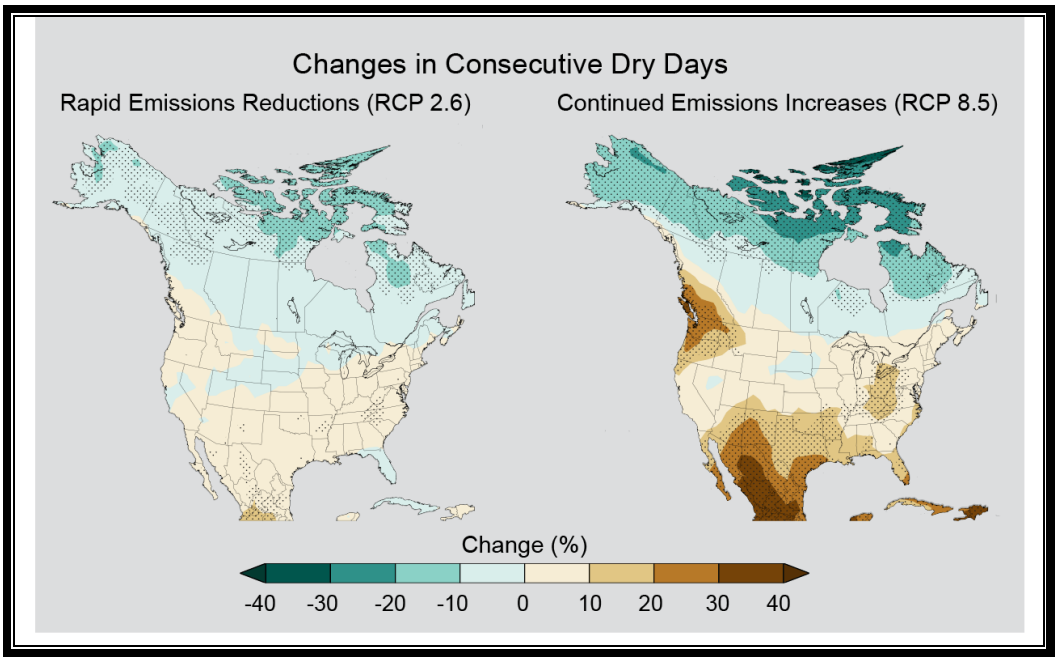

Figure 4-7 - Changes in Consecutive Dry Days

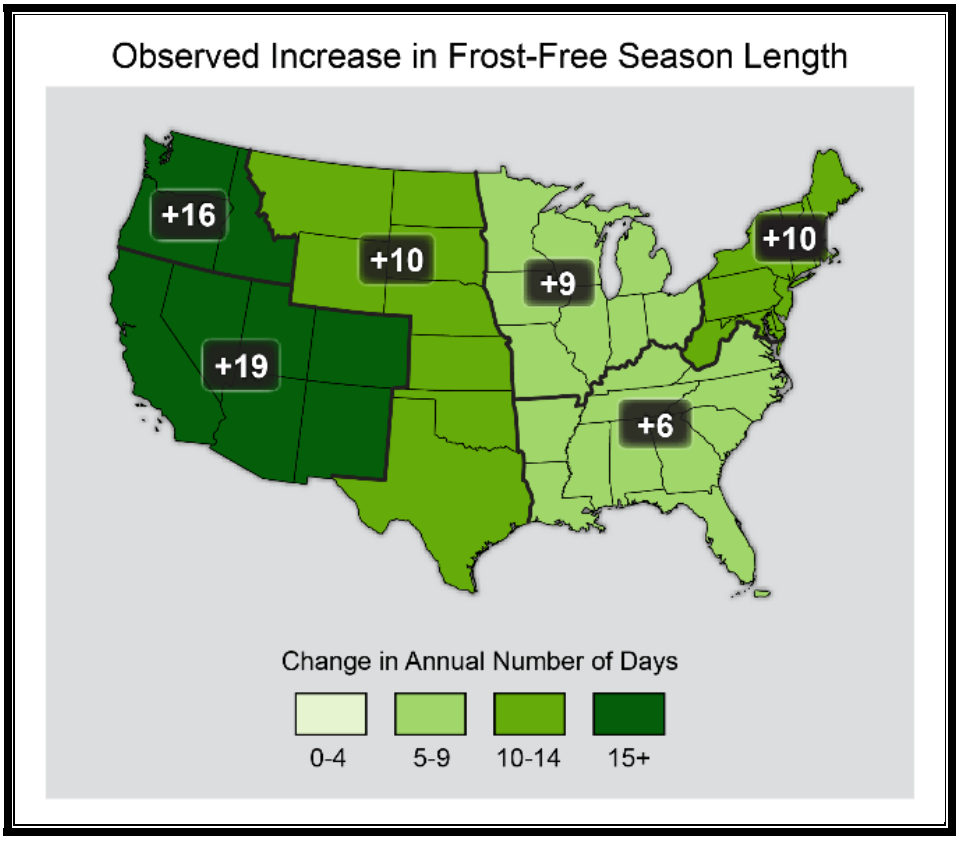

Figure 4-8 - Observed Increase in Frost-Free Season Length

Few actions can be taken in the context of the erosion control to make the parcel more resilient to higher

future sea levels, higher rainfall runoff flows, overall dryer conditions, and higher temperatures. In the event of additional deterioration of the wall from extreme wave impacts, resulting in new cracks in the wall, this erosion control design can be adapted by placing additional flowable fill (or an equivalent granular, free-draining material) in new voids.

5 Pertinent Data

No survey data was used for this study. The National Historic Preservation Act Section 106 Report (Sastre 2005) was supplied by the Sponsor. Geotechnical investigation files were provided by The Florida Geological Survey. A comprehensive topographic survey will be acquired at the outset of the DI phase to develop plans and specifications. This survey will provide the latest topographic features (erosion damage), easements, structures, utilities, and streets, etc.

6 Civil Design

The site quantities (cut, fill, etc.) were determined by estimating quantities from visual inspection. An estimated 50 cubic yards (cy) each of flowable fill (or an equivalent granular, free-draining material) and topsoil was estimated. The topsoil will be made up of select fill from within the same county. Weep holes will be spaced approximately ten feet apart, and will have a diameter of four inches. Approximately 5 cy of shotcrete will be needed, which can be placed at low tide to avoid the cost of dewatering. Exact quantities will be determined during DI, as well as addressing any existing drainage pipes that may need to be protected during construction.

7 Real Estate

This project may require the acquisition of a temporary real estate easement for construction access for any work to be done by barge. The parcels needed for this access appear to be owned by the State of Florida. No temporary real estate easement will be required for staging areas, as all staging will be performed within the St. Francis Barracks Complex. The real estate needs and staging areas are shown in Figure 6-1. This project is located within the City of St. Augustine, use of existing public city streets for transportation of construction equipment and material will be feasible.

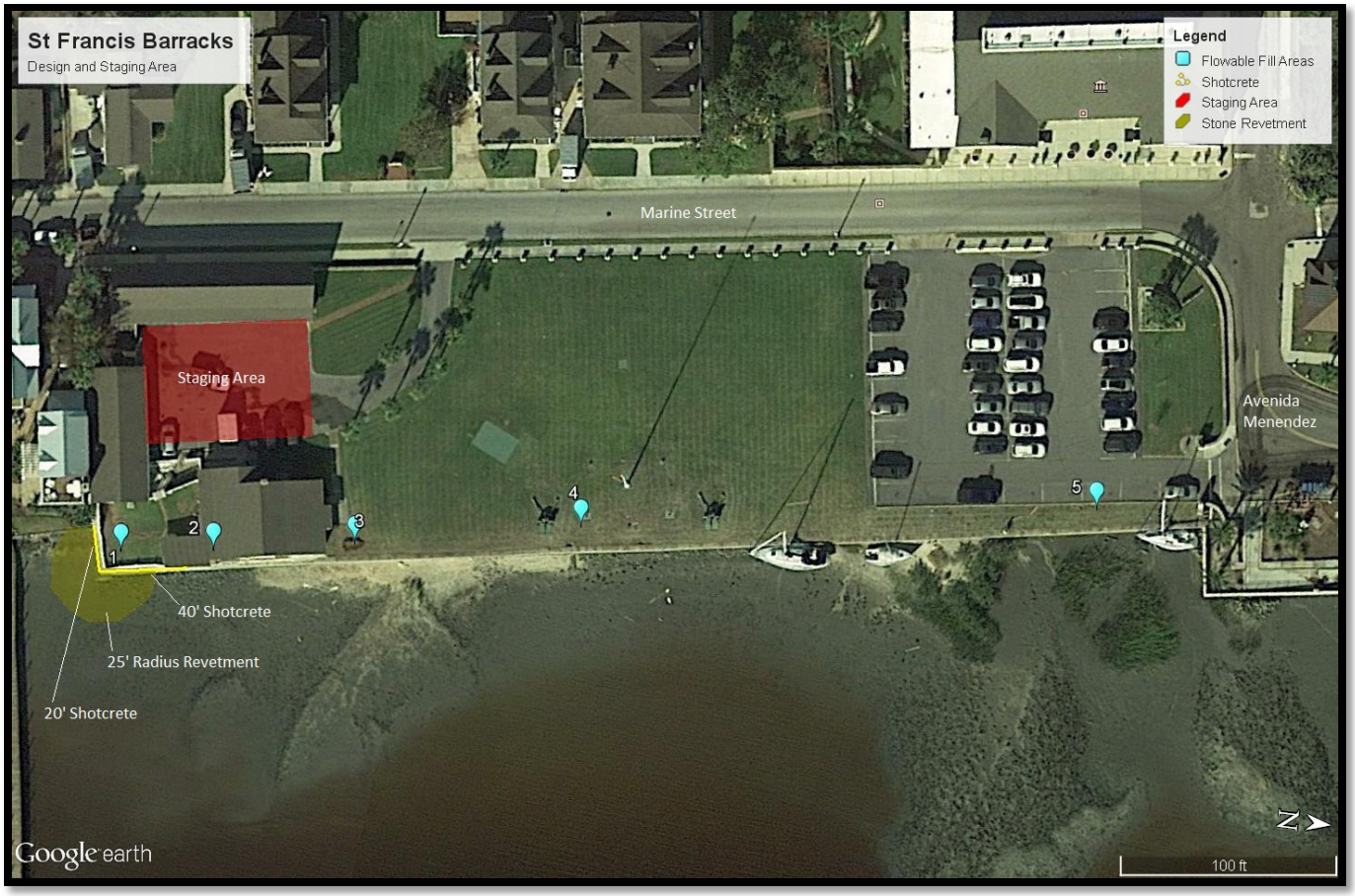

Figure 6-1 - Access Roads, Staging and Storage Areas

8 Relocations

Utilities located in the vicinity of the project were identified by design engineers during on-site inspections. A storm sewer inlet was noted, as shown in Figure 6-2. This pipe as with all utilities and infrastructures around and beneath the JAG building, will be protected prior to placement of flowable fill. If the removal and/or relocation of any utilities is determined during design, the same size and type of material would be utilized in the relocation of a utility; however, none is expected.

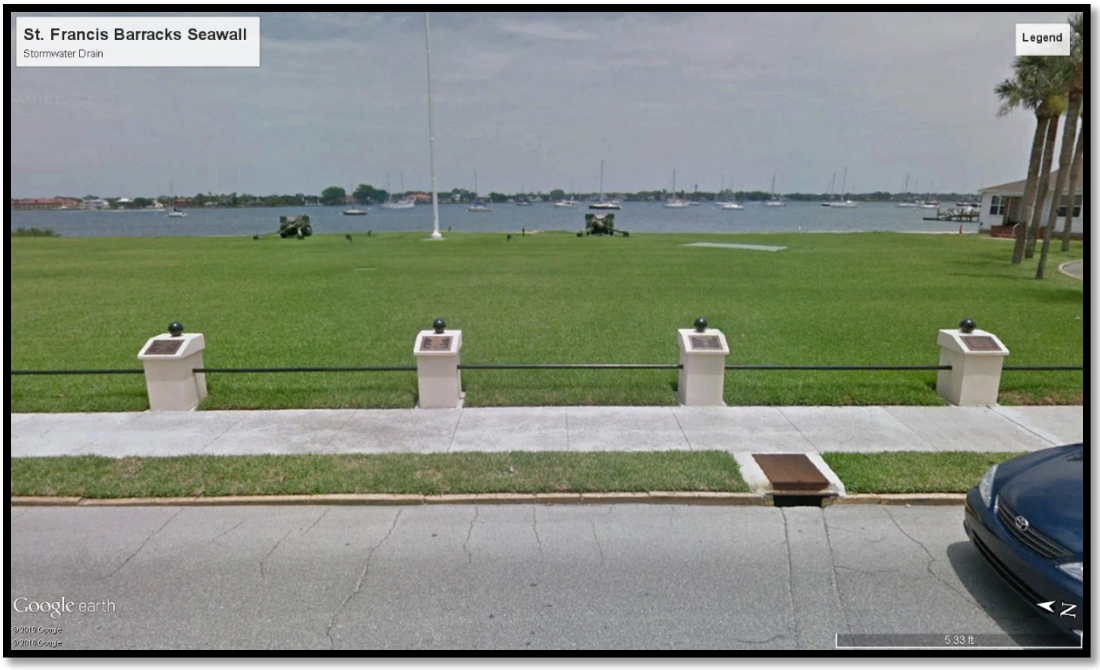

Figure 6-1 – Storm Sewer Inlet

9 Structural Requirements

It was determined by the USACE technical team that a structural analysis will not be necessary as part of this design. During the site visits, the engineering team members agreed that no global structural failure or instability was observed. There was no rotation or change to alignment, only minor isolated settlement of the granite capstone (this includes the undated repairs to the southeast corner). Additionally, the wall has withstood over a century of storms thus far, indicating structural stability.

The Recommended Plan requires minimal excavation, as discussed in Section 2.4, to minimize contact with the wall and avoid the risk of changing the soil loading behind the wall.

10 Electrical and Mechanical Requirements

There are no electrical or mechanical elements in this project design.

11 Hazardous, Toxic, and Radioactive Waste (HTRW)

Details of the environmental analysis performed for this Feasibility Study is presented in Chapters 2 and 5 of the Main Report. Phase I and Phase II Environmental Site Assessments (ESAs) will be completed during the Design and Implementation (D&I) phase.

12 Construction Procedures and Water Control Plan

Runoff and erosion control measures will also be put in place during construction to minimize erosion of the minor amount of excavation, and all adjacent land that may have been stripped of vegetation. Bypass pumping (the temporary diversion of water from a specific working area (dewatering) to a selected location), will be performed in accordance with the approved Erosion Control Plan and within construction limits. The Erosion Control Plan will be developed by the construction contractor and reviewed by USACE prior to beginning construction.

13 Design and Implementation (D&I) Data and Tasks

The schedule and budget provided for this feasibility study did not allow for inclusion of data collection or engineering analysis. The following engineering data collection and detailed engineering analyses have been determined to be necessary to ensure a proper design, and must occur during the D&I Phase:

- Subsurface data collection, including ground penetrating radar (GPR) tests and core borings;
- Design level survey collection, including utility locations and adjacent parcel owners; A detailed structural survey;
- Seepage analysis to determine proper weep hole spacing and medium for proper drainage;
- A Value Engineering Study;
- Determine setback distance from wall so construction vehicles will have no impact;
- Phase I Environmental Site Assessments (ESAs);
- Assessment of infrastructure (drainage system, utilities, and JAG building) and determination of protection measures;
- Analysis of appropriate grout and flowable fill (or an equivalent granular, free-draining material) mixes;
- Construction sequence; and
- Final Design of Alternative, preparation of Plans and Specifications.

14 Residual Engineering Risk

As discussed in Section 11, the budget and schedule required for this project did not allow for any engineering analysis scope, and all new data collection and engineering analyses were deferred to the DI phase. Deferring necessary engineering analysis to the DI phase greatly increases the risk of design changes and cost increases. These were discussed and captured in the cost risk register. Some of the risks include but are not limited to:

- Risk of discovering greater void space than currently estimated, resulting in possible extensive excavation adjacent to the wall, requiring more construction and fill material, and a possible structural

analysis;

- Risk of spacing, size, and number of weep holes increasing;
- Risk of determining larger stone size needed to protect corner; and
- Risk of other unknown site condition requiring a design or material change.

There is greater additional risk if the required engineering data collection and analyses listed in Section 11 are not performed. Some of these risks include but are not limited to:

- Risk of seepage path to weep holes blocked and the design failing if no seepage analysis is performed; and
- Risk of the seepage analysis being incorrect if no subsurface data is collected.

15 Operation and Maintenance

After construction completion, the recommended regular maintenance includes annual weep hole clearing, inspection for voids and damaged infrastructure following a hurricane event, and annual structural inspections of the wall. This design will not require any operation.

16 Schedule for Design and Construction

The schedule for design and construction is provided in the separate Cost Appendix.

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