

Department of Naval Architecture and Ocean Engineering  
Ocean Engineering Capstone Design

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## Flooding and Erosion in Fort Castillo de San Marcos

by  
Team Monuments Pals

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## Executive Summary

Fort Castillo de San Marcos, a Fort located half a mile north of St. Augustine, is dealing with flooding throughout the year. On the Fort grounds, the flooding issues are seen at the south wall, parking lot, north wall and the National Park headquarters located about a quarter mile northwest of the Fort. The solution to the flooding must uphold the cultural integrity of the site and accommodate sea level rise over a 50 year project duration as well as conditions of a 1% annual exceedance probability storm.

The National Park rangers requested that the team provide a variety of solutions at varying cost ranges. While brainstorming ideas, several limitations were set, such as the north wall could not be raised due to neighbors' requests, the mangroves could not be destroyed due to environmental considerations, and the south wall also could not be raised because of how it channeled water to the city. Finally, none of the solutions could affect the Fort's infrastructure due to archeological concerns.

The park rangers expressed interest in incorporating green solutions into the problem, but were not opposed to gray solutions as long as it meets the previous constraints. For the National Park headquarters, it was concluded that only flood proofing could be used to limit flood damage. This was due to a variety of factors such as cost, location of the headquarters, and environmental restrictions. Within the compound of the north wall, the major focus was incorporating green solutions to the already present mangroves. Between the north and south side of the Fort, there was a major push to have at least one green solution.

After several ideas were brainstormed, the decision matrix was formed based on eight different characteristics. These characteristics are design life, cultural integrity, aesthetics, cost, functionality, timeline, simplicity, and environmental effects. All locations of the Fort were ranked against one another by using a weighted scale.

Though several options are provided, the best options were selected using a decision matrix. The decision matrix proved that a breakwater and beach nourishment for the north wall, a revetment for the south wall and an earthen levee for the parking lot were the best options. All of the calculations were based on the foundation of a design water level of 7.33 feet above NAVD88 for the revetment and the earthen levee, and 9.33 feet above NAVD88 for the breakwater due to its distance offshore.

For the north wall it was calculated that two 80 foot breakwaters would be the most successful regarding sediment accretion and wave energy dissipation. Additionally the breakwaters would be separated by 80 feet and be 80 feet from the shoreline (to the center of the breakwater). Between the separation and distance from the shoreline, the breakwaters would be successful in limiting flooding during storm surges. In addition to breakwater, a beach nourishment process could be used for flood limitation. The nourishment would create a berm 7 feet above NAVD88 with a 1 on 10 slope. Between the berm and the slope, the nourishment would also be able to limit flooding and break down wave energy.

For the south wall, the revetment and earthen levee were the two best options for flood limitation. The revetment was 298 feet from north end to south end and came off the seawall 26.5 feet. The size of the revetment would help prevent flooding and break down wave energy. The earthen levee was the smallest consideration and rather than preventing flooding worked to channel the water away from the parking lot and downtown area. At 6 feet high and having a 12 foot diameter the earthen levee would be successful in flood deviation.

Between the four major considerations, the breakwater held the highest price while the earthen levee was considered the most cost effective. Through estimating area and weight, the table on the next page shows the overall price if all four projects were considered. The projects do not depend on one another allowing for selection of one or all of them.

Overall, there are many solutions to each of the individual problems in their respective areas. Based off design criteria, the best solutions to the north flooding problem are breakwaters, a beach nourishment or continued wet and dry flood proofing, and for the south flooding problem a revetment or an earthen levee. In consideration of the eight different characteristics from the decision matrix, the breakwater for north flooding and the earthen levee for the south wall would be the primary selections offered for client. The customer will have the option of any combination.

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# Introduction

## ***1.1 Background***

Fort Castillo de San Marcos, shown in Figure 1, has been facing issues with chronic flooding in its National Park Headquarters due to a low seawall on the north side of the fort. Similarly, the southern sea wall of the fort is significantly lower than that of the connecting city seawall leading to flooding of the city and fort's parking lot. In extreme storm events, these problem areas are the weak points in the city's flood proofing and leave the city at risk for storm damages, as discussed in the following section of text.



**Figure 1: Aerial View of Fort Castillo (Google Maps, 2019)**

## ***1.2 Flooding Orientation***

Extreme storm events threaten the functionality of the National Park Headquarters to the north and the parking lot to the south. Figure 2 shows the southern road leading to the entrance of the parking lot. The right portion of Figure 2 shows the result of 3 feet of flooding due to the storm surge associated with Hurricane Irma in 2017.



**Figure 2: Avenida Menendez road stoplight located at the entrance to the parking lot. The right image shows the results of 3 feet of flooding during Hurricane Irma in 2017 (Kaijuzilla)**

Similarly, Figure 3 shows the water level in the park's building headquarters for both Hurricane Matthew and Irma. Hurricane Matthew resulted around 28 inches of flooding at the northern buildings associated with Fort Castillo and approximately 17 inches during Hurricane Irma.

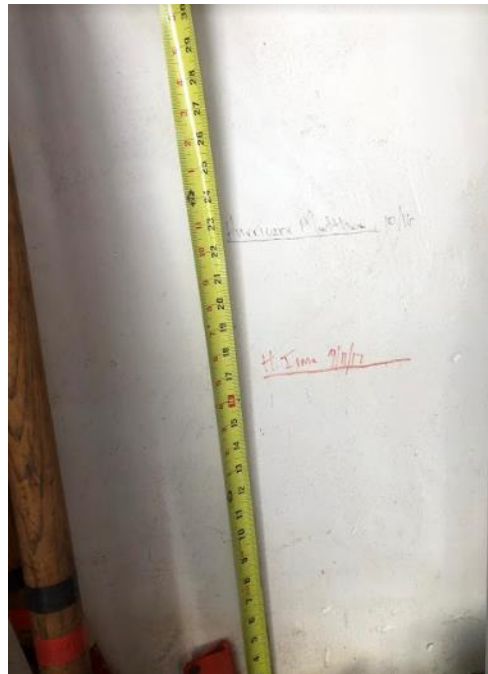


**Figure 3: Measurements of the water levels at the National Park's Headquarters due to Hurricanes Matthew and Irma (Picture taken during site visit).**

The NOAA Sea Level Rise Viewer tool allowed for an aerial orientation according to different levels of water surge above Mean Higher High Water level (MHHW), as shown in Figure 4 and Figure 5.

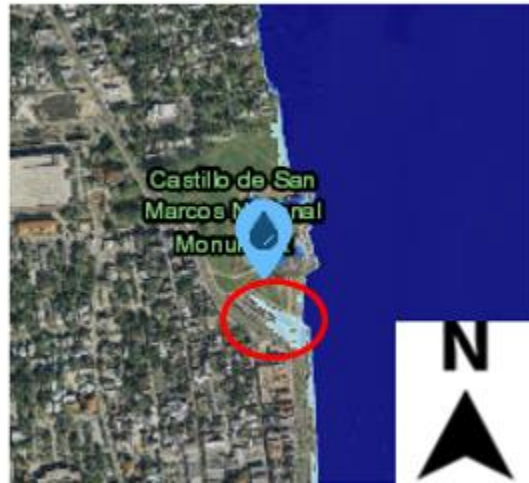


**Figure 4: Aerial view of the potential flooding areas due to a storm surge of 2 feet above MHHW (NOAA Sea Rise Viewer tool).**

Figure 4 shows that a significant portion of the southern parking lot flooded whereas the northern Headquarters is not impacted with a storm surge of 2 feet above MHHW.



**Figure 5: Aerial view of the potential flooding areas due to a storm surge of 3 feet above MHHW (NOAA Sea Rise Viewer tool).**

Figure 5 shows significant flooding impact on the southern parking as well as the city streets along the seawall with a storm surge of 3 feet above MHHW. Flooding also reached the northern headquarters location and neighboring area.

### ***1.3 Purpose behind the Project***

The importance of this project is to ensure the ability for the public to be able to benefit from touring and observing Fort Castillo de San Marcos as a National Park. This involves an emphasis on ensuring that the National Park is usable through maintaining a functioning parking lot and National Park Headquarters. The functioning parking lot allows the public the ability to access the monument as well as provides a steady source of economic income to fund the maintenance and operations involved with keeping the fort running. Similarly, the National Park rangers are unable to properly care for the fort without a fully functioning Headquarters. The surrounding city of St. Augustine also plays an important role in supporting tourism involved with the national monument. The fort is directly across hotels, bars, restaurants, and nightlife activities that help promote public accessibility to the fort. Providing the fort with increased flood prevention solutions ensures that these facilities remain functional in order to provide the public an opportunity to visit this historical site.

### ***1.4 Historical Significance of Fort Castillo***

Castillo de San Marcos served as the northernmost outpost of the Spanish's colonial New World Empire. The fort began construction in 1672 to protect the city of St. Augustine from Pirate threats as well as dissuade any attempts by England to forcefully colonize there territory. During the War of the Spanish Succession, England occupied the fort but eventually abandoned it after burning the town of St. Augustine. In this way, the fort serves to symbolically link the original colonial background of the city to its rebirth from the ashes. Through the course of its lifetime, the Fort Castillo has served as a constant beacon of protection and is attached to the spirit of St. Augustine. Today it still serves as a historic monument as the best kept example of a Spanish colonial fortification and is heavily connected to the town's vibrant history and delightful atmosphere.

### ***1.5 Problem Statement***

The problem to be addressed is the Castillo De San Marcos' issue of chronic flooding along their northern and southern walls. These walls protect their National Park headquarters as well as city structures. The solution must also uphold the cultural

integrity of the site and accommodate sea level rise over a 50 year project duration as well as conditions of a 1% annual exceedance probability storm. The National Park association has asked for several different solutions at varying price ranges.

# Site Conditions

## 2.1 Site Visit

The team visited the site from September 13 – 15, 2019 with bearable weather and ample time to take necessary data. Prior to walking the Fort Castillo grounds, a HOBO tide gage was placed on a pier at the St Augustine Municipal Marina a mile south of the Fort as shown in Figure 6. This was able to give important data leading into design water level value.

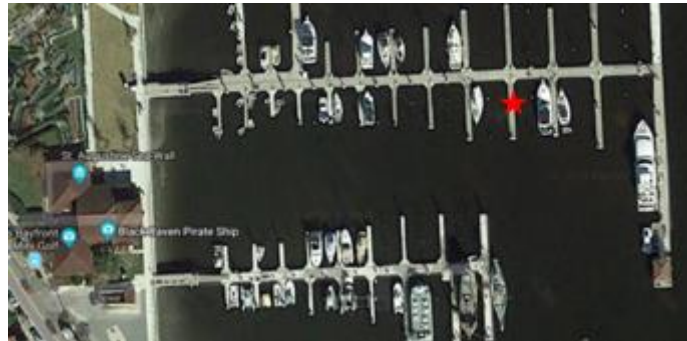


**Figure 6: Location of the Tide Gauge (Google Maps, 2019)**

Figure 7 displays the depth of the sensor below the water during the field deployment, which was used to measure the tide changes in St Augustine. Figure 8 compares the pressure change and the temperature change. These values also allowed the team to observe the daily changes in these water properties.

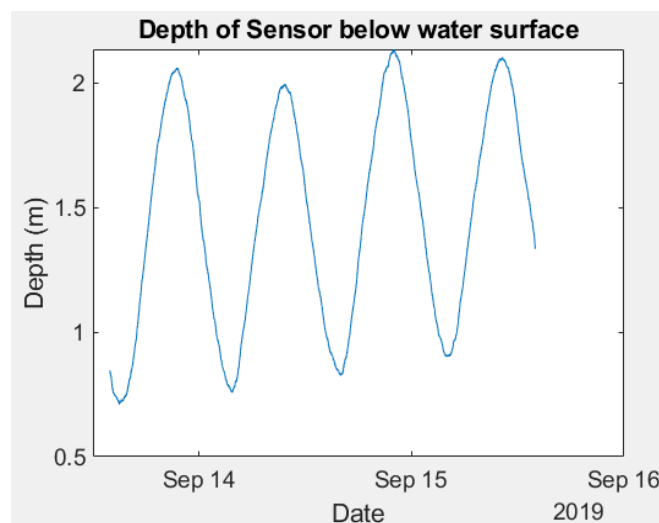


**Figure 7: Depth of Sensor below surface**

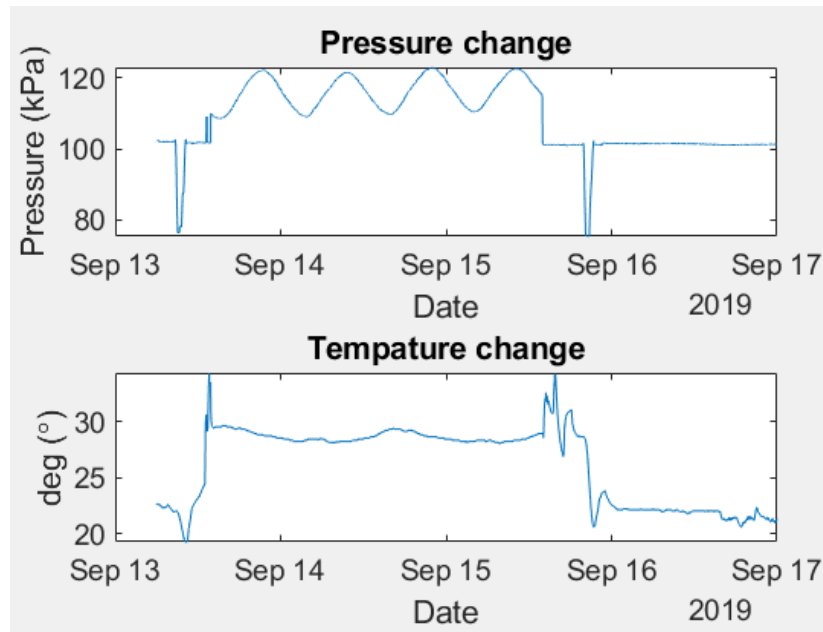


**Figure 8: Pressure Change and Temperature Change during Site Visit**

Following the tide gage deployment, the team met with several National Park Rangers, including Shane Compton, James Crutchfield and Steven Roberts. They explained that they wanted a variety of options which they could mix and match based on different variables, such as cost and duration of construction. Additionally, the rangers asked for a 50 year project duration and for the project to be able to withstand a 1% annual exceedance probability storm. Following this discussion, the team visited the first vulnerable area.

The north wall was noticeably lower than the rest of the fort's coastline, and was susceptible to overtopping by waves, especially during major storms. The rangers explained how they noticed the mangroves and hoped to continue their growth along the seawall. Additionally, they pointed out that they had considered raising the wall but had anticipated possible legal issues with neighbors to the north, shown in Figure 9. If the north seawall was raised it would channel water around it, leading water to move to the north. This could possibly lead to legal action against the park.



Figure 9: Map of the North Wall and Headquarters (Google Maps, 2019)

While walking from the north wall to the south wall, six different measurements were taken to find water temperature, salinity, conductivity, dissolved oxygen and pH as shown in Table 1. The locations of these measurements are shown in Figure 10. These values were able to guide the group on understanding mangrove growth and where their growth will be the most successful.

Table 1: Water Characteristics

| Location | Time | Water Temperature (°C) | Salinity (ppt) | Conductivity (umhos/cm) | Dissolved Oxygen (mg/L) | pH   |
|----------|------|------------------------|----------------|-------------------------|-------------------------|------|
| #1       | 1448 | 29.6                   | 38.92          | 58489                   | 5.13                    | 7.9  |
| #2       | 1517 | 29.6                   | 38.64          | 58112                   | 5.64                    | 8.03 |
| #3       | 1525 | 29.6                   | 38.02          | 57416                   | 4.81                    | 8.08 |
| #4       | 1528 | 30.8                   | 37.39          | 56972                   | 5.17                    | 8.11 |
| #5       | 1533 | 29.7                   | 38.76          | 58273                   | 5.11                    | 8.1  |
| #6       | 1550 | 30.3                   | 38.84          | 58489                   | 4.92                    | 8.11 |



**Figure 10: Location of Measurements (Google Maps, 2019)**

Though the north wall was a major concern, the National Park service's main goal was not to stop flooding on the national grounds entirely, but to stop flooding at the headquarters, a group of buildings approximately 300 yards directly west of the wall. The issue was that the sea level would rise so high that it would reach the headquarters building and flood the building itself and surrounding structures. The rangers explained they are raising structures and currently using temporary flood walls placed in front of openings, as shown in Figure 11, but they wanted more effective solutions. Therefore, in addition to providing solutions for the north wall, solutions to prevent the flooding headquarters are also expected to be provided.



**Figure 11: Headquarters Raised Structures and Flood Walls**

The south wall flooding was different from the north. The rangers explained that the south wall would flood, leading to flooding to the parking lot, and finally to downtown St Augustine, shown in Figure 12. They also stated that the parking lot was a vital part of their income, and its downfall would cause major economic effects. Additionally, unlike the north wall, the south wall did not have any mangroves but did have oyster growth and a breakwater as shown in Figure 13. Though the breakwater did lower the wave energy, it only did so at low tide. During high tide, the breakwater was almost completely submerged, lowering wave energy slightly and not limiting flooding at all, as shown in Figure 13. The breakwater also did nothing for sediment transport. With only two small inlets, there was no sediment being built up over time.



**Figure 12: Map of South Wall and Parking Lot. Arrows signify direction of flood flow. (Google Maps, 2019)**



**Figure 13: South Wall at High and Low Tide**

For both the north and south walls, the park rangers explained they wanted a variety of solutions. They showed preference for green solutions, but were also open to grey solutions. They hoped to keep the existing mangroves and oysters at the north and south walls respectively, and encouraged solutions that would increase their growth and overall numbers. Finally they preferred the most economic options but anticipated that the best solutions may be more expensive.

## **2.2 Met Ocean Data**

Data were collected and graphed using the NOAA tide gauge located in Mayport, Florida. The location of tide gauge was approximately 60 miles away from the project area, additionally the tide gauge is located on the open ocean in comparison to the project area which is in a protected river. Figure 14 and Figure 15 below depict the average wave height and wind speed near the project area. As shown, the peak wave height is 3.5 meters and the top wind speed of 18-20 m/s. The majority of winds are northeasters, causing a majority of erosion problems at the project area.

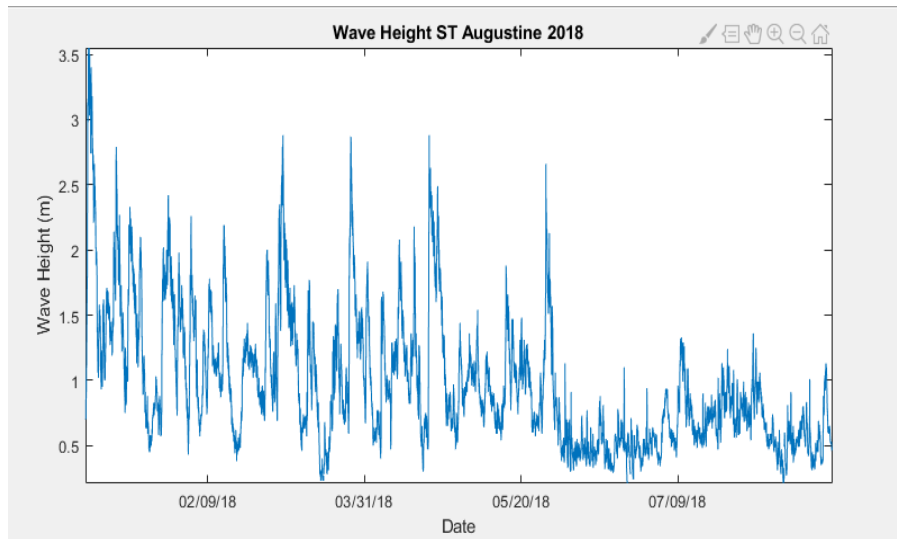


**Figure 14: Wave Height data in St Augustine**

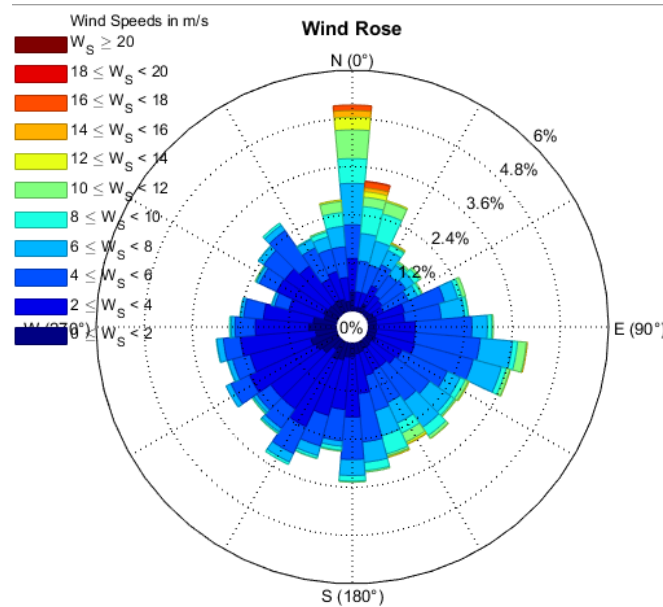


Figure 15: Wind Rose of St Augustine

## 2.3 Sea Level Change

The mean sea level trend is +0.01 inches per year which is equivalent to a change of 0.625 feet over 50 years above NAVD88. This approximation was estimated using Figure 16 below showing the estimated sea level change over 100 years representing low, intermediate, and high extrapolations of sea level rise. The intermediate green curve was used as a conservative estimate of the design water level. The sea level rise was combined with a 50 year storm tide surge of 4.25 feet for a design water level of +4.875 above the mean sea level, and +6.1 feet above the NAVD88 datum.

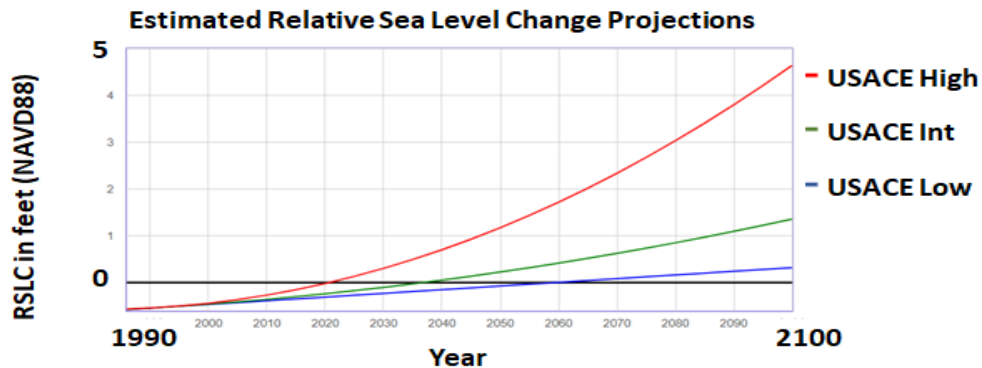


Figure 16: USACE Multiple Sea Level Change Projections

## 2.4 GIS Bathymetry

Using an open source program called QGIS allowed for the development of a basic bathymetry of the water surrounding the north and south side of Fort Castillo. A digital elevation map of the Fort and the surrounding area was obtained from a US Army Corps of Engineers' LiDAR survey in 2016. This map was overlaid as a raster dataset in QGIS on the study area. The resulting topographical map is shown in Figure 17. Using the terrain profile plug-in for QGIS allowed for the ability to create transects over any area of concern, which provided the height and slope along the length of each individual transect. An example of this for both the north and south wall is shown in Figure 18. The results showed that the seafloor directly surrounding the seawall is located at -0.6 m with respect to NAVD88.

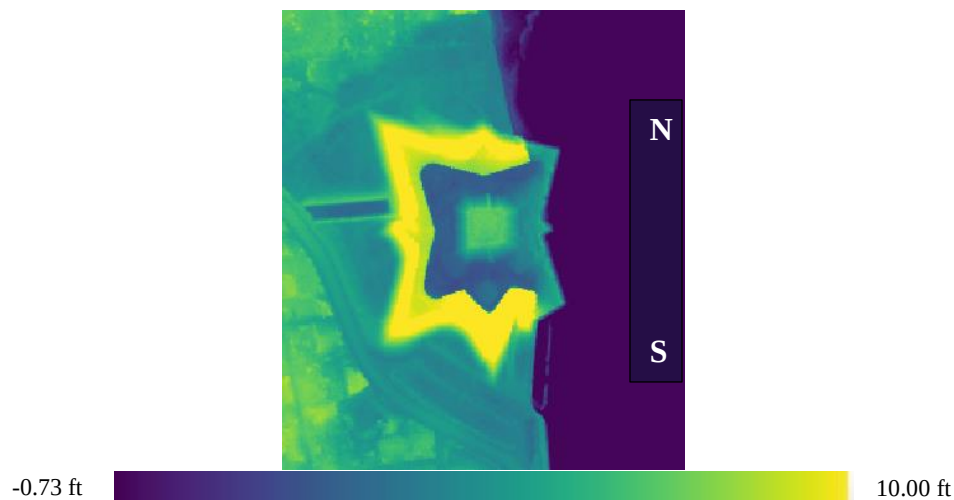


**Figure 17: GIS Bathymetry of Fort Castillo (QGIS, 2019)**

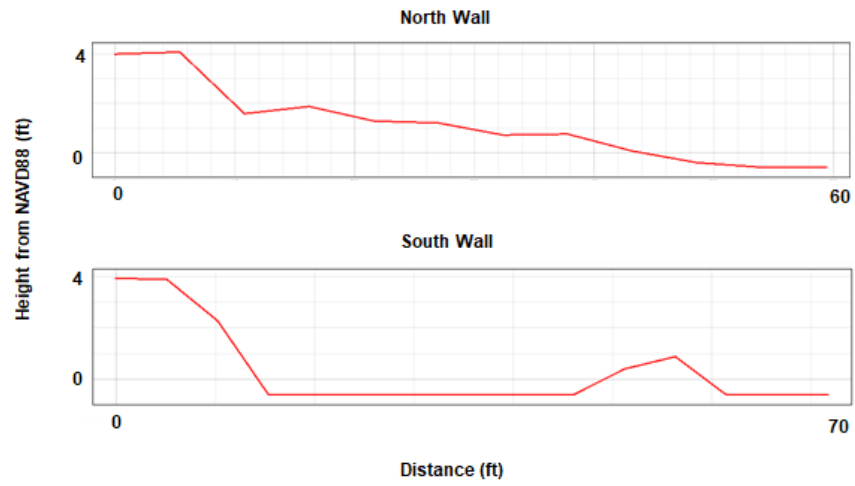


**Figure 18: GIS Profile view of North and South Walls (QGIS, 2019)**

# Preliminary Design Process

## 3.1 Design Criteria

The first step in the solution process was to develop a set of design criteria on which each design would be judged. Each member of the group developed separate criteria, which were all discussed and culled down to those seen in Table 2. These criteria were largely based on the needs of the customer discussed during the site visit. The criteria were ranked on a 5 point scale, with the requirements shown in the table. The requirements were chosen based on the knowledge and research done through March of this semester. Cost is still being evaluated in more detail. This may have an effect on the decision matrix as more knowledge is acquired.

**Table 2: Design Criteria**

|            | Options                                      |                          |                                          |                                                        |
|------------|----------------------------------------------|--------------------------|------------------------------------------|--------------------------------------------------------|
| 5 pt Scale | Design life                                  | Cost                     | Aesthetics                               | Cultural Integrity                                     |
| 4          | 50 years                                     | \$                       | Matches existing aesthetics + green      | No disruption to current ground or infrastructure      |
| 3          | 40 years                                     | \$\$                     | Matches existing aesthetics              | Disruption to non-historical infrastructure            |
| 2          | 30 years                                     | \$\$\$                   | Minimally disrupts tourism potential     | Minimal disruption to ground/infrastructure            |
| 1          | 20 years                                     | \$\$\$\$                 | Significantly disrupts tourism potential | Having to dig or disrupt current ground                |
| 0          | 10 years                                     | \$\$\$\$\$               | Severely disrupts tourism potential      | Destroy or change existing infrastructure              |
| 5 pt Scale | Functionality/Reliability (flood prevention) | Duration of construction | Simplicity                               | Effects on ecosystem/community                         |
| 4          | 7-9ft (meets 50yr storm)                     | 3 months                 | Easy to integrate                        | No effect                                              |
| 3          | 5-7ft                                        | 6 months                 | Requires minimal complex equipment       | Minimal sand transportation issues                     |
| 2          | 3-5ft                                        | 9 months                 | Average Design/Equipment required        | Medium sand transportation and wildlife issues         |
| 1          | 1-3ft                                        | 1 year                   | Requires complex design/equipment        | Severe sand transportation/displaces existing wildlife |
| 0          | <1ft                                         | 1 year 3 months          | Not Feasible                             | Flooding neighboring coastlines                        |

## 3.2 North Side Design Alternatives

Several design alternatives were considered for the north side area. These are discussed in greater detail below.

### 3.2.1 Breakwaters

One of the design alternatives to prevent erosion on the north side of the fort is a breakwater. Figure 19 is a real life example that is trying to be matched. It is a breakwater on the York River in Virginia and works to break down wave energy and create a tombolo. A tombolo, shown in Figure 20, is an example of sand accretion which extends the beach and limits erosion. Limiting erosion, sand accretion and the dissipation of wave

energy are three major pros to breakwaters. Having a huge formation of rocks piled up around a national monument could potentially impact the tourism in the area. Figure 21 represents arbitrary dimensions and locations of the breakwaters on the project site.



**Figure 19: Breakwater example (National Park Service Breakwater, 2012)**

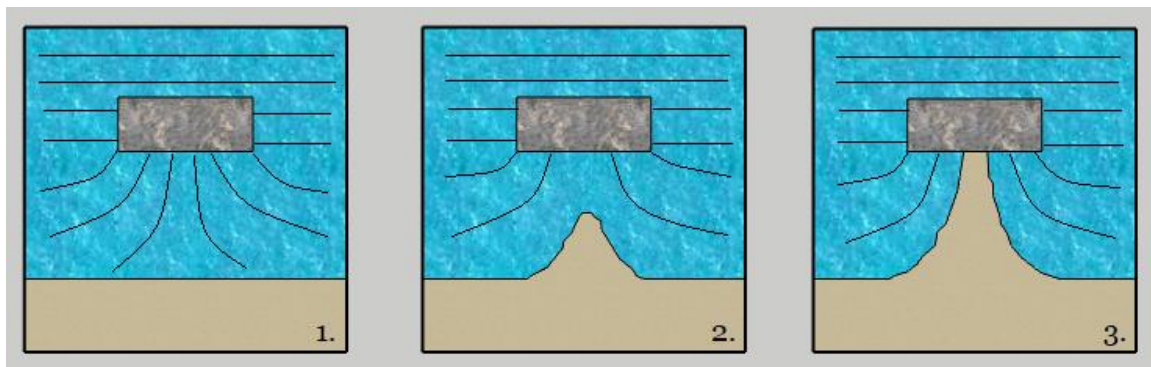


**Figure 20: Tombolo creation**

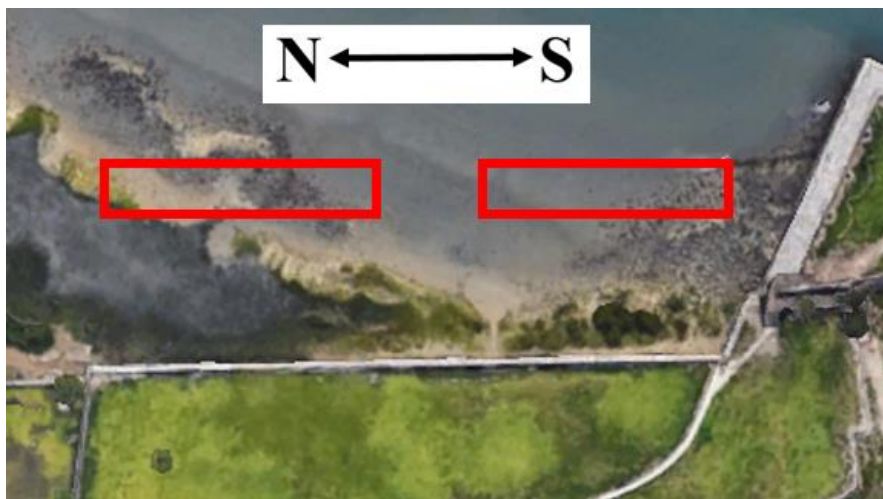


**Figure 21: North Wall Potential Breakwaters (Google Maps, 2019)**

### 3.2.2 Groin

The second design alternative for the north side is a groin, shown in Figure 22. A groin is a series of rocks piled perpendicular to the shoreline, and its purpose is to collect sediment moved by longshore transport. Groins have pros and cons. One of the pros is that based off the sediment transport, it accretes sand in the project area. However, groins have fell out of favor due to downdraft issues causing erosion in a different location.

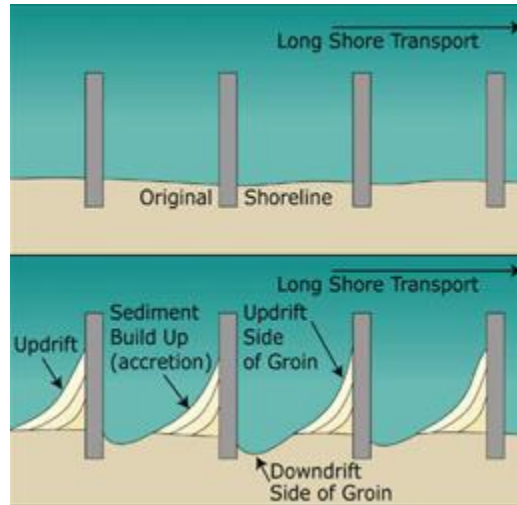


Figure 22: Example Groin (Google Images, 2019)

### 3.2.3 Headquarters Flood-proofing

One of the main focuses of the project was flood-proofing the headquarters on the north side of the project area. Traditional flood proofing tactics are extremely effective in limiting flooding of the building. Tactics such as placing vertical bulkheads with removable flood barriers as shown in Figure 23, and elevating the building as shown in Figure 24 produce efficacious results that will prevent further flooding to the building. While these tactics may be effective, they require manual labor for installation before heavy tides/storms/hurricanes. Building elevation may also be cost-prohibitive. Another tactic for protecting the headquarters deals with wet flood proofing. Wet flood proofing allows the water to enter the building, but it creates holes throughout the building to allow water to flow out. Figure 25 shows an example of wet flood proofing for the headquarters. Wet flood proofing can be advantageous because it is fixed, immovable, and does not require any manual labor before rising water levels. This tactic does allow

water to enter the building which could result in potential problems.



**Figure 23: Flood barriers outside Rickover Hall at USNA**

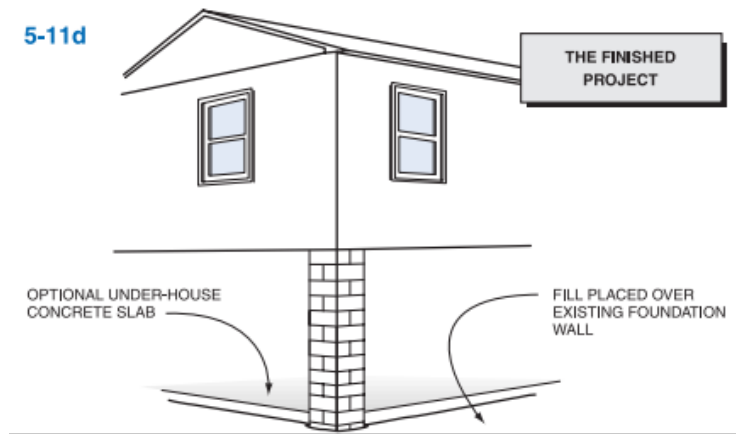


**Figure 24: Example of raised building (FEMA, 2019)**

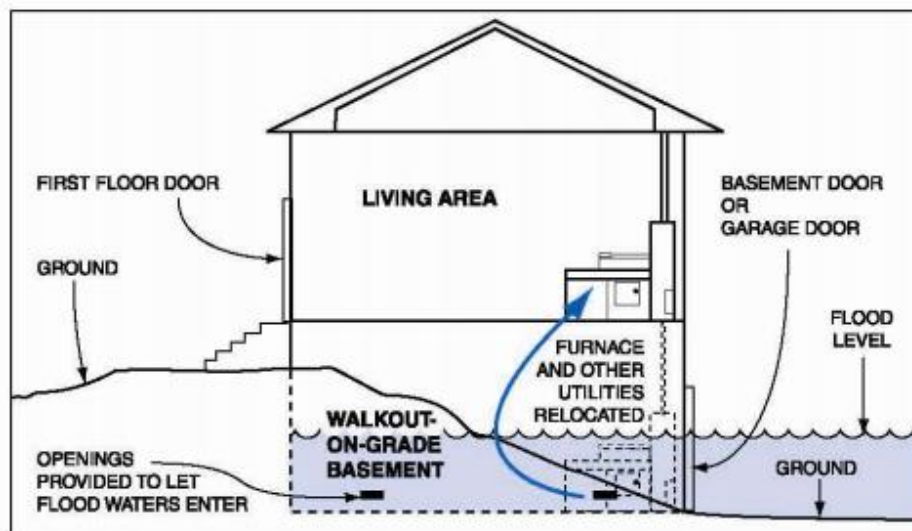


**Figure 25: FEMA Wet Flood-proofing Example (FEMA, 2019)**

### **3.2.4 Beach Nourishment**

One design alternative to prevent flooding at the north side of the fort is beach nourishment. Beach nourishment is the process of adding sediment to an existing beach front in order to establish an equilibrium beach profile. In this case, the beach nourishment would be used in order to create a beach berm for the purpose of flood prevention. An example of the type of beach nourishment discussed can be seen in Figure 26. Sand's natural tendency to reach an equilibrium profile to match the existing coastal environment gives it a number of unique properties in terms of coastal protection. According to the Bruun Rule, as sea level rise occurs over time, the beach's equilibrium profile will rise as well. While this results in the retreat of the shoreline, it helps to naturally balance out sea level rise over time. A diagram depicting the Bruun Rule can be seen in Figure 27. Properly designed beach nourishment systems are highly effective at preventing flooding. During large storm events, the resulting storm surge causes the beach profile to be eroded to a magnitude that varies based on the severity of the storm. This means that by extending the beach berm far enough landward, large storms will have no effect beyond the area of beach that they erode. This also means that during large storm events, a significant portion of the beach front will be lost and can only be regained through natural sediment accretion over time or by nourishing the beach once again. A diagram depicting the beach front's response to a significant storm event can be seen in Figure 28.

Beach nourishment is an ideal design alternative in terms of flood prevention, environmental consciousness, and social impact. The beach front would offer an aesthetically pleasing solution. The addition of a significant beach front adjacent to the fort would provide added social benefits, as it would likely help to boost tourism and could be used for recreational purposes by visitors. Beach nourishment would offer an inherent factor of safety in its design as beach fronts are naturally occurring and typically not prone to failure. The downfall to beach nourishment is that for it to be effective in flood prevention the beach front must be maintained over time, potentially requiring the process to be repeated in order to overcome sediment erosion should it occur.

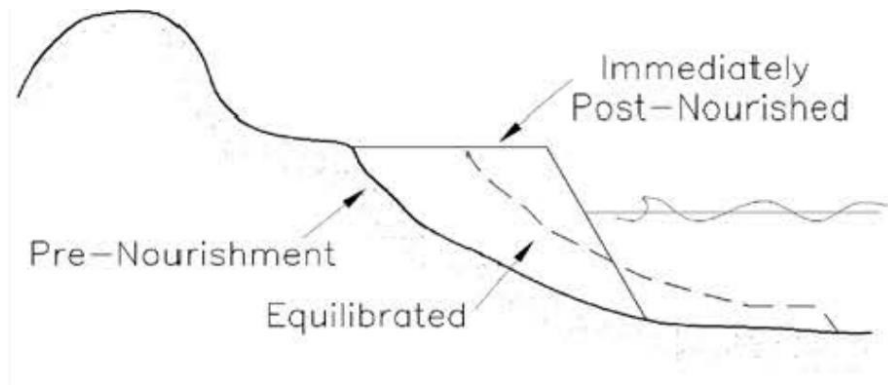


Figure 26: Example Beach Nourishment Design (Anonymous, 2002)

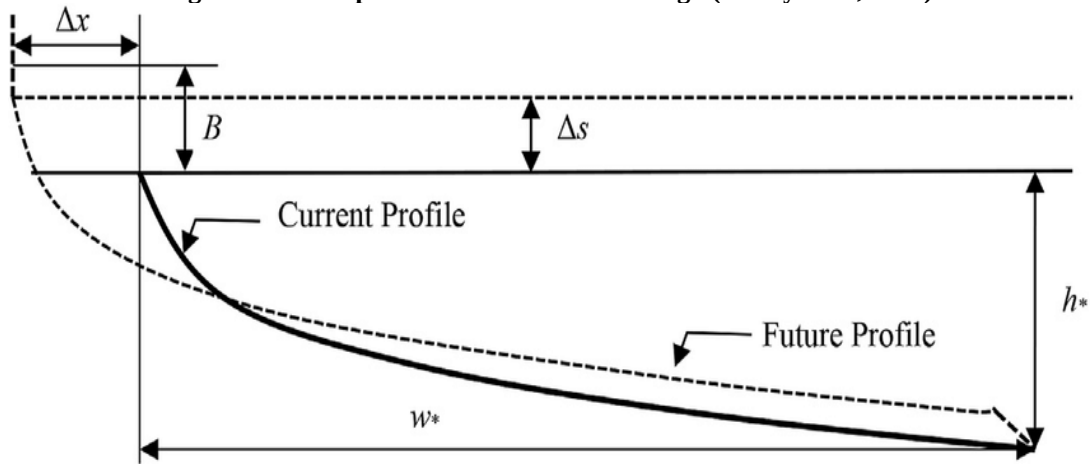


Figure 27: Diagram of Bruun Rule (Anonymous, 2002)

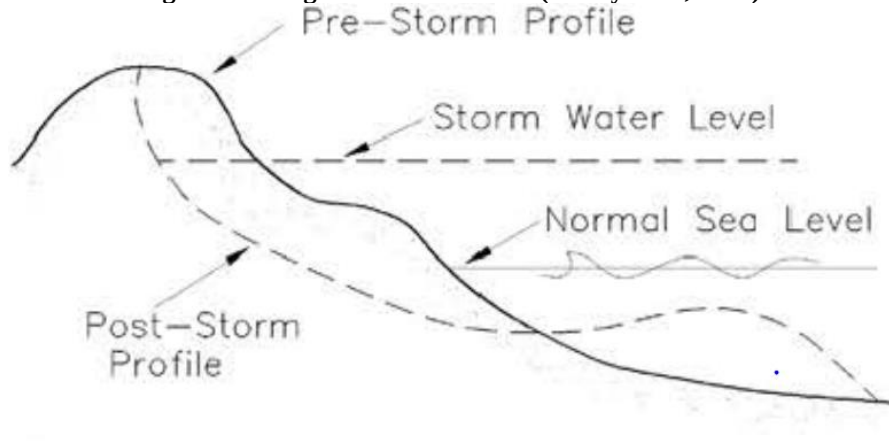


Figure 28: Diagram of Beach Front Response to a Storm (Anonymous, 2002)

### 3.3 South Side Design Alternatives

Several design alternatives were considered for the south side area. These are discussed in greater detail below.

### 3.3.1 Earthen Levee

The earthen levee is the first of three solutions focusing on the parking lot on the south side of the fort. This design proposes constructing a levee that extends from the seawall to the upper edge of the parking lot, running adjacent to the sidewalk of the parking lot. The levee will act as a barrier to shield the parking lot from incoming flood water, effectively damming it off and keeping water in the grassy field adjacent to the fort where it will not cause any significant damage. An initial conceptual sketch of the levee created in Sketch Up can be seen in Figure 29 and Figure 30.

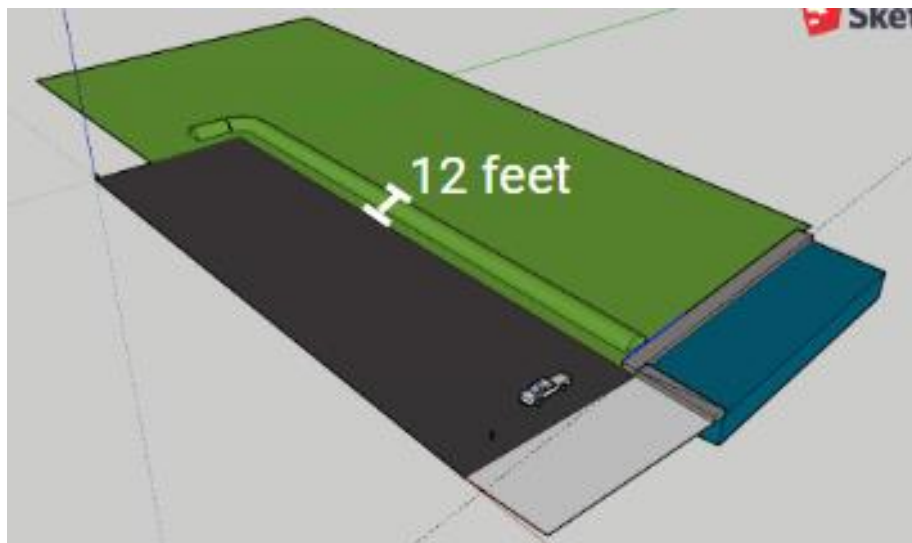


Figure 29: Earthen Levee Design View 1

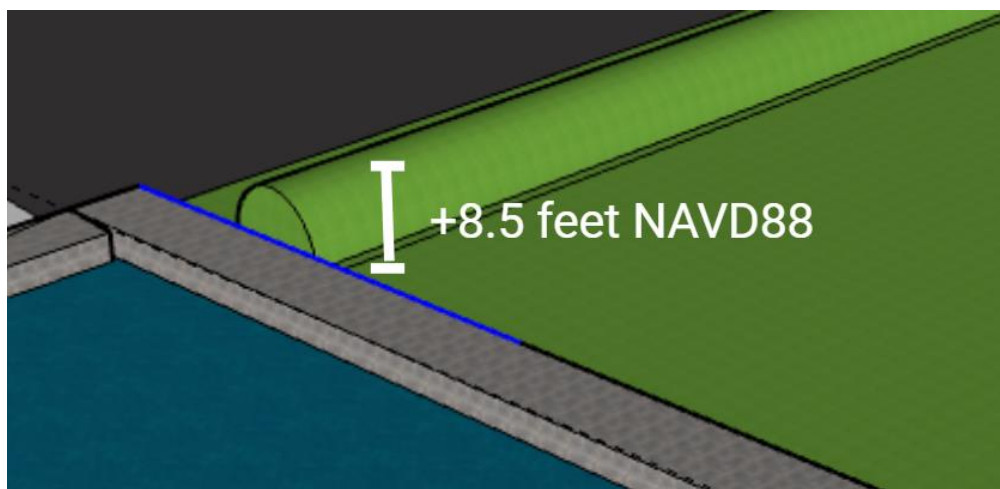


Figure 30: Earthen Levee Design View 2

The initial proposed dimensions of the levee are a height of +8.5 feet in reference to NAVD88 and a width of twelve feet. The elevation of the existing ground ranges from +4.38 feet to +5.10 feet, both in reference to NAVD88, along the proposed area of construction. This crest elevation will guarantee protection of the parking lot with respect to the proposed design water level, +6.1 feet NAVD88, while overestimating in order to allow a built in factor of safety to account for any wave run-up that will occur. The final design and construction of the proposed levee will be done in accordance with the U.S. Army Corps of Engineers manual for the design and construction of levees. (Anonymous, 2002)

The earthen levee is an ideal design in terms of effectiveness, invasiveness and its potential environmental and social impact. A levee offers a great deal of flood protection for a structure that is quite adaptable. The design life of a levee is substantial, and it can easily be removed if a different design is needed towards the end of the design life. A levee is a green structure and will likely have no negative effect on, and could potentially improve, the aesthetic of the fort. A levee offers an inherent factor of safety in its design as levees in nature are not prone to catastrophic failure and any repairs to a levee are relatively cheap and easy. The downfall of a levee is that it will inhibit the natural flow of the water cycle, and could potentially cause flooding in the event of heavy rainfall. This is by no means a disqualifying factor, but the runoff will need to be taken into account.

### ***3.3.2 Vertical Bulkhead***

The vertical bulkhead is the second of three solutions focusing on the parking lot on the south side of the fort. This design proposes constructing a bulkhead, fitted with removable flood barriers, that extends from the seawall to the upper edge of the parking lot, running adjacent to the sidewalk of the parking lot. With the flood barriers deployed, the bulkhead will act as a barrier to shield the parking lot from incoming flood water, effectively damming it off and keeping water in the grassy field adjacent to the fort where it will not cause any significant damage. An initial conceptual sketch of the bulkhead created in Sketch Up can be seen in Figure 31.

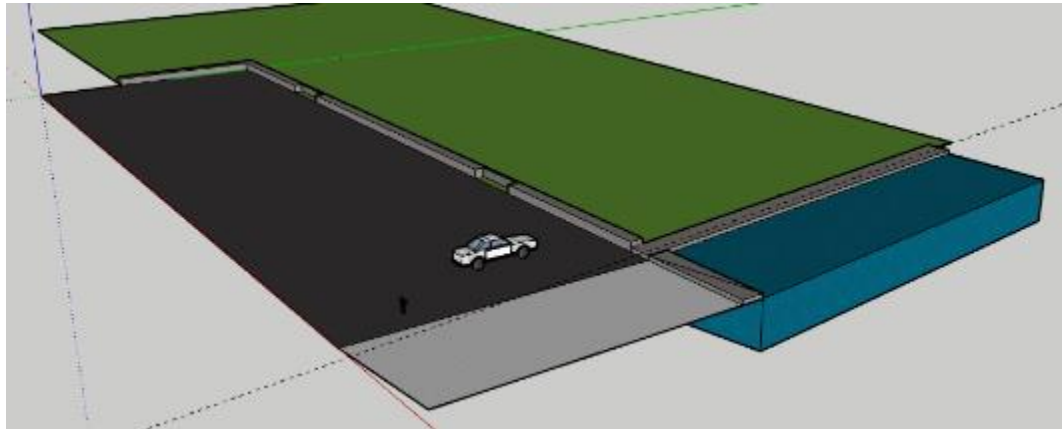


**Figure 31: Vertical Bulkhead Design**

The initial proposed dimensions of the bulkhead are a height of +8.5 feet in reference to NAVD88. This elevation, like that of the earthen levee, will guarantee protection of the parking lot with respect to the proposed design water level, +6.1 feet NAVD88, with a built in factor of safety to account for wave activity.. The proposed removable flood barriers will be similar in design to those employed in Rickover Hall at the United States Naval Academy, seen previously in Figure 23. The final design and construction of the proposed bulkhead will be done in accordance with the U.S. Army Corps of Engineers manual for the design of coastal revetments, seawall, and bulkheads. (Anonymous, 2002)

The bulkhead is an ideal design in terms of flood protection, but it lacks in aesthetic and reliability. The removable flood barriers solve the earthen levee's issue with impeding natural water flow, as they will be removed except in extreme conditions that would result in the city flooding. The bulkhead is more invasive than the levee, not allowing for any adaptation over time. The hard grey nature of the seawall does little for the environment and takes away from the natural aesthetic appeal of the proposed construction area. One large issue with vertical bulkheads is that when they do fail, failure is catastrophic, and in this particular event that will result in huge amounts of flood damage to the city of Saint Augustine.

### 3.3.3 Raised Parking Lot

The raised parking lot is the last of the three solutions focusing on the parking lot on the south side of the fort. This design proposes building up the parking lot with a solid base in order to elevate it six feet upwards. The raised parking lot will act as a barrier to shield the city from incoming flood water, effectively damming it off and keeping water in the grassy field adjacent to the fort where it will not cause any significant damage. An initial conceptual sketch of the raised parking lot created in Sketch Up can be seen in Figure 32.

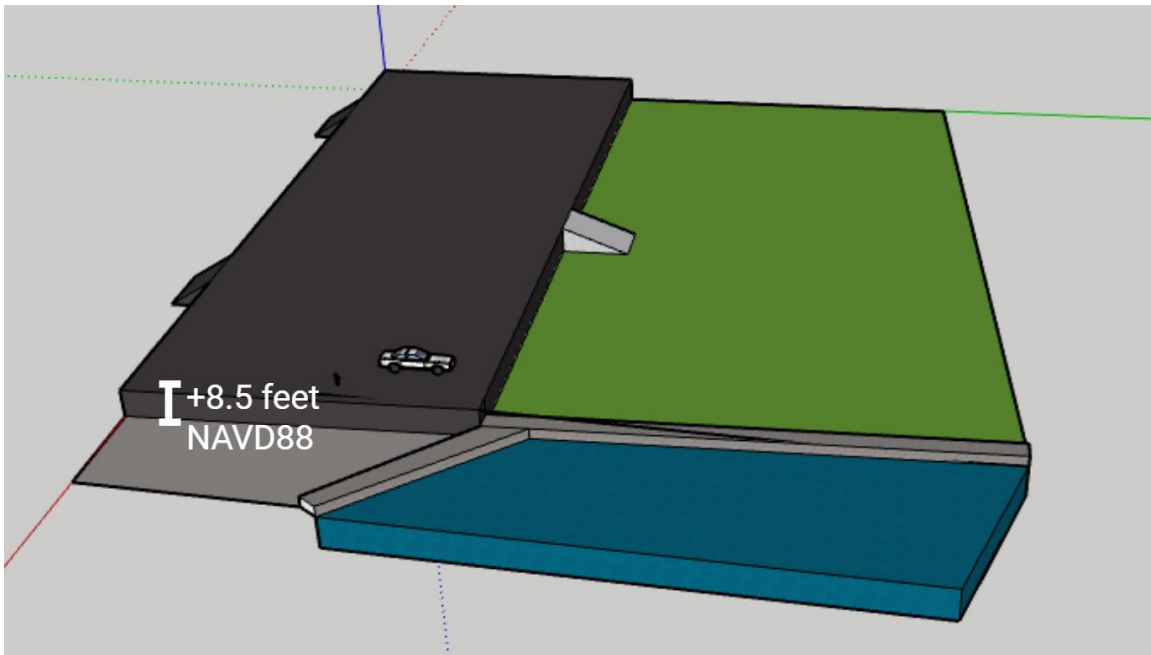


Figure 32: Raised Parking Lot Design

The initial proposed height of the parking lot is +8.5 feet in reference to NAVD88. This crest elevation will guarantee protection of the parking lot with respect to the proposed design water level, +6.1 feet NAVD88, while adding a built in factor of safety to account for any wave activity. As this design is not common practice for the proposed application of coastal engineering, there is no existing manual by which the design and construction will be governed.

While the raised parking lot is highly effective in flood prevention and quite durable, it has significant downsides in most other areas. As stated previously, a raised

parking lot is not common practice for use in coastal engineering, and thus would ideally be avoided if other alternatives prove to be acceptable. The raising of the parking lot would be a highly invasive process, requiring a great deal of construction and offering no potential for future development at the end of its design life. While it does provide the needed level of flood protection, raising the entire parking lot would prove to be unnecessary over construction, and thus costly. The hard grey nature of the design would detract from the area's natural appeal and impede the view from the fort to the city and vice versa. The raised parking lot shares the levee's issue of impeding natural water flow, but to a potentially worse degree, as rain runoff from the raised parking lot could cause unnecessary flooding.

### 3.3.4 Non-Living Revetment

From the earlier discussion of site conditions and orientation that the south seawall has an ineffective breakwater due to its low elevation at high tide as shown in Figure 33. Therefore, a possible alternative would be to build a revetment in place of this breakwater. A revetment would involve a design toe, slope, and splash in accordance to the wave energy hitting the southern wall along the lines of the design taken from the Coastal Engineering Manual, shown in Figure 33.

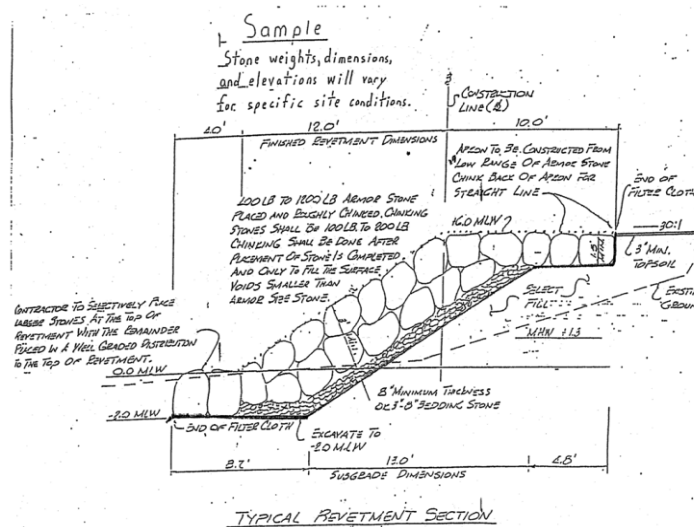


Figure 33: Revetment Example (Coastal Engineering Manual, 2019)

Building a revetment according to this design is a simple alternative with a long engineering design life and durability. Regarding this project's design criteria, an aspect of the design that scored low on the 5 point scale is the potential cost, and it is a relatively grey solution to the flooding problem, as shown in Figure 34.

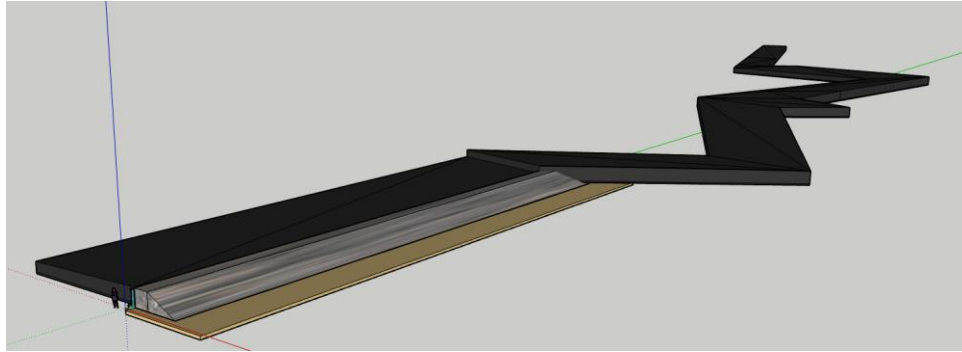


**Figure 34: Isometric view of Non-living Revetment**

### ***3.3.5 Living Revetment***

Adding a living aspect to the revetment offers a more green solution rather than a completely grey alternative. The design would include several gaps along the proposed non-living revetment where sand would be allowed to travel in and out with the tide. Planting mangroves and other shoreline protective plants would help hold the sediment between the revetment and seawall more securely. A Sketch Up model of this potential alternative is shown in Figure 35.

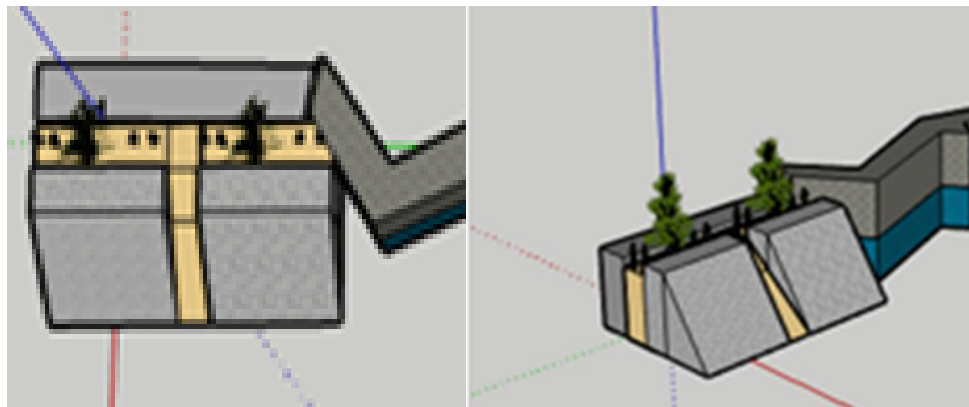


**Figure 35: Living Revetment**

This living revetment alternative raises some potential problems. This design is

slightly more complex than the solid non-living revetment. The complexities of the living alternative comes through the possible inclusion of sand nourishment during construction, which adds further expenses to an already expensive design. Similarly, the cost of maintaining this design needs to be analyzed because sediment movement would be a free flowing factor as time progressed and would require possible further sediment nourishment.

### 3.2.6 Decision Matrix

The decision matrix for the north and south side can be seen below in Figure 36. Each design criteria was given a relative weighting based on the needs of the customer. For the north side, the breakwater beat out the groin with an Object Engineering Process (OEP) score 0.35 higher. The breakwater will be the solution to the north side. For the issue of the parking lot, the earthen levee design significantly beat out the other alternatives with an OEP score of 3.50. For the seawall along the south side, the revetment without a living shoreline beat out the revetment with a living shoreline by 0.15. The earthen levee and revetment without a living shoreline will be the solutions of choice going forward. Beach nourishment beat out headquarters fortification, but both options are viable.

| Criteria           | Relative Weighting | Options    |       |                            |                   |                        |               |                         |                               |                                |
|--------------------|--------------------|------------|-------|----------------------------|-------------------|------------------------|---------------|-------------------------|-------------------------------|--------------------------------|
|                    |                    | North Side |       |                            |                   | South Side/Parking Lot |               |                         |                               |                                |
|                    |                    | Breakwater | Groin | Headquarters Fortification | Beach Nourishment | Raised Parking Lot     | Earthen Levee | Wall Around Parking Lot | Revetment w/ Living Shoreline | Revetment w/o Living Shoreline |
| Design Life        | 0.15               | 4          | 4     | 2                          | 5                 | 4                      | 4             | 4                       | 4                             | 4                              |
| Cultural Integrity | 0.15               | 4          | 4     | 5                          | 4                 | 3                      | 4             | 1                       | 4                             | 4                              |
| Aesthetics         | 0.05               | 3          | 2     | 2                          | 5                 | 3                      | 4             | 1                       | 4                             | 3                              |
| Cost               | 0.20               | 2          | 3     | 4                          | 2                 | 4                      | 3             | 3                       | 1                             | 1                              |
| Functionality      | 0.20               | 4          | 2     | 2                          | 4                 | 2                      | 3             | 4                       | 4                             | 4                              |
| Timeline           | 0.05               | 3          | 3     | 4                          | 3                 | 2                      | 4             | 3                       | 3                             | 3                              |
| Simplicity         | 0.10               | 2          | 3     | 4                          | 3                 | 3                      | 4             | 4                       | 3                             | 4                              |
| Environmental      | 0.10               | 4          | 2     | 5                          | 4                 | 3                      | 3             | 3                       | 3                             | 4                              |
| 1<br>OEP scores:   |                    | 3.3        | 2.95  | 3.45                       | 3.65              | 3.10                   | 3.5           | 3.05                    | 3.15                          | 3.30                           |

Figure 36: Design Matrix

# Advanced Design Process

## 4.1 Design Wave Height

The design wave height associated with a 50 year storm event was found using wind fetch calculations. Wind fetch calculations were done according with the guidance laid out in the Coastal Engineering Manual (CEM), and all equations used come from this source. The wind gust data used comes from the National Data Buoy Center Station SAUF1, which is 3.90 miles southeast from the site of interest. The gusts reported are the five second maximum over a one hour period. Gust data was used from September 1986 to December 2008, and the data can be displayed as a mean and standard deviation plot as seen in Figure 37.

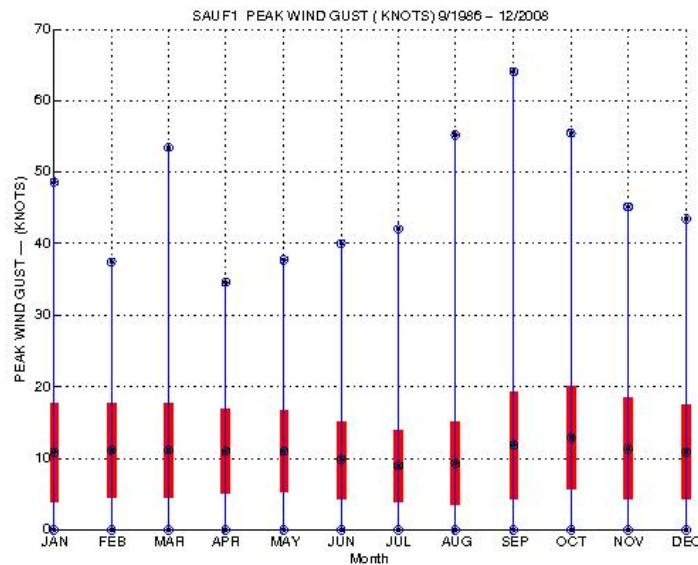


**Figure 37: Mean and Standard Deviation Plot of Wind Gusts from 1986-2008 (National Data Buoy Center, 2020)**

The top ten gust events over the time period were plotted and extrapolated using a logarithmic curve out to a 50 year return period, the raw data used can be found in Table 3 and the curve itself can be seen in Figure 38. The 50 year design wind gust was found to be 124.8 feet/second.

**Table 3: Data of Top Ten Gust Events between 1986-2008**

| Ranking | Return Period, Tr<br>(years) | Gust (ft/s) |
|---------|------------------------------|-------------|
| 1       | 22.0                         | 107.4       |
| 2       | 11.0                         | 93.0        |
| 3       | 7.3                          | 92.6        |
| 4       | 5.5                          | 89.8        |
| 5       | 4.4                          | 81.6        |
| 6       | 3.7                          | 75.7        |
| 7       | 3.1                          | 72.8        |
| 8       | 2.8                          | 70.5        |
| 9       | 2.4                          | 67.1        |
| 10      | 2.2                          | 63.3        |

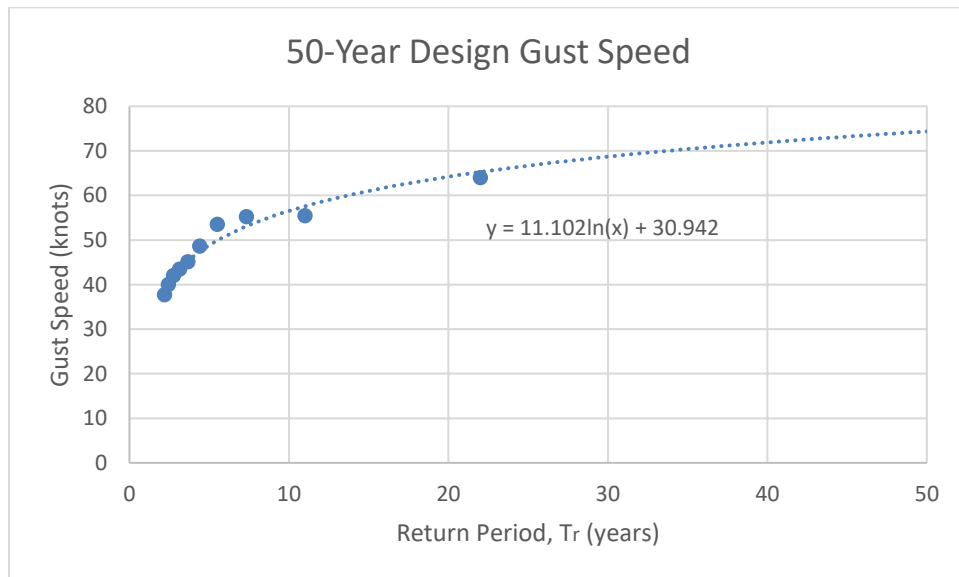


**Figure 38: Logarithmic Extrapolation of 50-Year Design Gust Speed**

The fetch distance upon which the wind wave calculations needs to be considered was determined to be 8,530 feet. The distance measured using the Google Earth ruler tool can be seen in Figure 39.

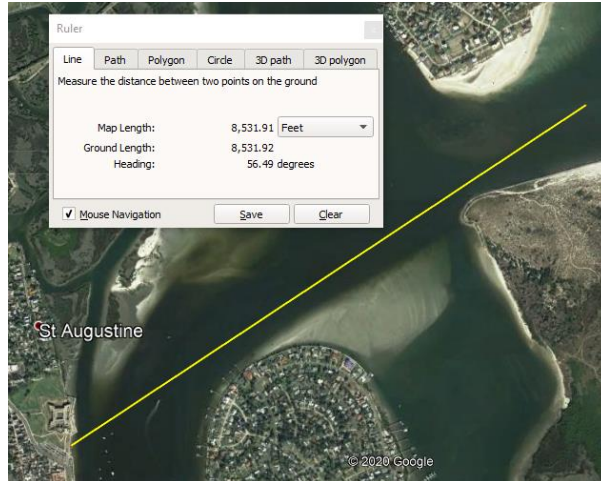


**Figure 39: Fetch Distance (Google Earth, 2020)**

The fetch distance was not taken further out of the mouth into the open ocean because the assumption was made that any existing waves coming through the opening of the mouth would be negligible at the site of interest due to refraction. The 5 second sustained wind gust was then extrapolated to its equivalent 3600 second sustained wind speed using Figure 40 from the Coastal Engineering Manual.

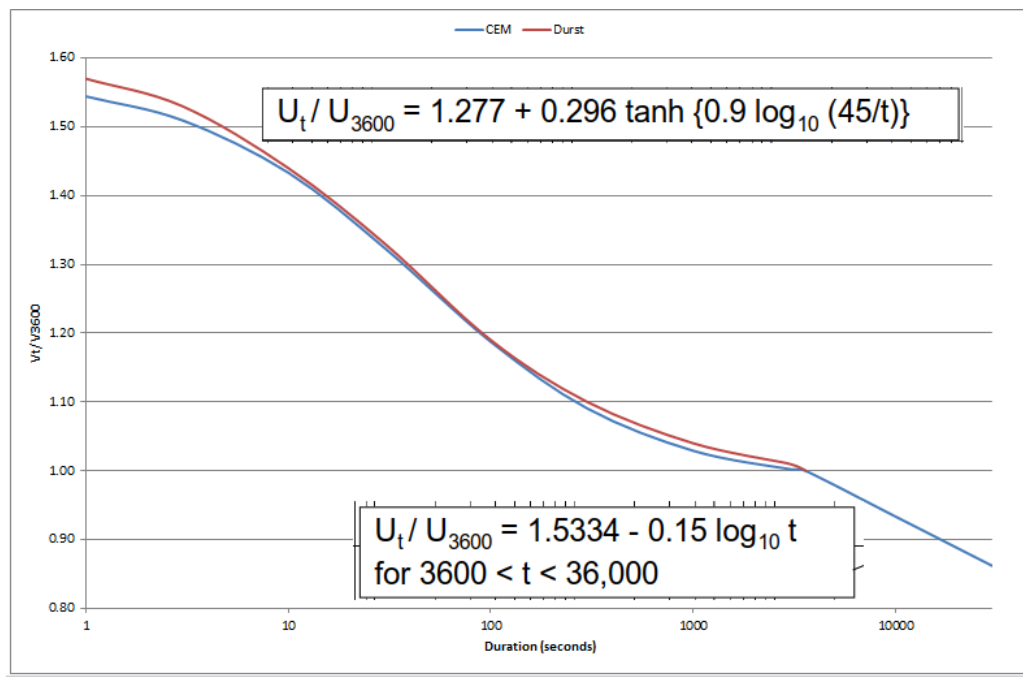


**Figure 40: Comparative Ratio of Wind Speed Durations (CEM, 2020)**

The critical duration for forming maximum wave growth,  $T_{x,u}$ , was found using equation 1

$$T_{x,u} = \frac{x^{0.67}}{u^{.34} * g^{.33}} \quad 1$$

where  $x$  represents the fetch distance in meters,  $u$  represents the 3600 second sustained wind speed in m/s, and  $g$  represents the gravitational constant  $9.81 \text{ m/s}^2$ . The Coastal Engineering Manual requires that fetch wind wave calculations be done in metric units, so for this portion wind speed was converted to m/s and metric units were used, then upon completion converted back to English units to be consistent with the rest of this report. The wind speed sustained at  $t_{x,u}$ ,  $u_t$ , was found using Figure 40. Next the drag coefficient,  $C_D$ , was found using equation 2

$$C_D = 0.001(1.1 + 0.035u_t) \quad 2$$

where the drag coefficient has no units. The friction velocity,  $u_*$ , was found using equation 3

$$u_* = \sqrt{C_D} u_t \quad 3$$

The significant wave height caused by the wind over the fetch,  $H_{mo}$ , was found using equation 4

$$H_{mo} = \frac{u_*^2}{g} \left[ 0.0413 \left( \frac{gx}{u_*^2} \right)^{\frac{1}{2}} \right] \quad 4$$

The primary wave period,  $T_p$ , associated with the calculated significant wave height was found using equation 5

$$T_p = \frac{u_*}{g} \left[ 0.651 \left( \frac{gx}{u_*^2} \right)^{\frac{1}{3}} \right] \quad 5$$

The significant wave height was found to be 2.56 feet and the primary wave period was found to be 2.05 seconds. The fetch wind wave calculation done in accordance with the listed equations can be seen in Table 4.

**Table 4: Fetch Wind Wave Calculations**

| Variable   | Metric                | English      |
|------------|-----------------------|--------------|
| $x$        | 2600 (m)              | 8530 (ft)    |
| $u_5$      | 38.04 (m/s)           | 124.8 (ft/s) |
| $u_{3600}$ | 25.65 (m/s)           | n/a          |
| $t_{x,u}$  | 2048 (s)              | n/a          |
| $u_t$      | 25.90 (m/s)           | n/a          |
| $C_D$      | $2.01 \times 10^{-3}$ | n/a          |
| $u_*$      | 1.16 (m/s)            | n/a          |
| $H_{mo}$   | 0.78 (m)              | 2.56 (ft)    |
| $T_p$      | 2.05 (s)              | n/a          |

## 4.2 Revetment and Breakwater Design

Advanced designs for the revetment and breakwater solutions are discussed in further detail below.

### 4.2.1 Stone Design

Significant wave height was used in the following series of equations in order to establish a design crest elevation for both the proposed revetment and breakwater. Two methods were applied to find significant wave height due to a lack of buoy data available in the project area. The fetch method, as described in section 4.1, resulted in a significant wave height of 2.56 feet for both the north location of the breakwater and south location of revetment. The other method applied to find the significant wave height,  $H$ , was using the depth limited method driven by equation 6

$$H = 0.6d \quad 6$$

where  $d$  is the depth from NAVD88 to the ground. For the north location of the breakwater depth is equal to 2.06 feet. The south location of the revetment has a depth of 0.6 feet. Both the north and south locations depth limiting significant height was compared to the fetch method significant wave height in order to determine the most conservative design stone weight and stone diameter for both designs.

The Hudson Formula was used in order to calculate the stone weight,  $W$ , required based upon significant wave height, and shown by equation 7

$$W = \frac{W_a H^3}{K_d \Delta^3 \cot(\alpha)} \quad 7$$

where  $W_a$  is the unit weight of stone is assumed to be at 165 lb/ft<sup>3</sup>,  $K_d$  is the stability coefficient which is commonly assumed to be 2 for breaking waves,  $\Delta$  is the submerged specific gravity of stone equal to 1.58,  $\cot(\alpha)$  is the cotangent of the design structure slope of 2, and  $H$  is the significant wave height (feet). Using the calculated stone weight, the stone diameter,  $D_n$ , was found through equation 8

$$D_n = \left( \frac{W}{W_n} \right)^{\frac{1}{3}} \quad 8$$

Results comparing the different significant wave height calculations and its effects on stone weight and stone diameter are shown in Table 5.

**Table 5: Comparison of Different Design Stone Calculations**

| Method              | H (ft) | W (lbs) | $D_n$ (ft) |
|---------------------|--------|---------|------------|
| Depth at Breakwater | 1.9    | 73      | 1          |
| Depth at Revetment  | 1.18   | 17      | 0.47       |
| Fetch               | 2.56   | 207.38  | 1.08       |

Table 5 shows that the significant wave height due to fetch is the most limiting, and therefore, will be the most conservative design dimensions for both the breakwater and revetment stone weight and sizes. To add a factor of safety the design drawing the team decided to round the stone weight and diameter to the follow values listed in Table 6.

**Table 6: Factor of Safety Design Stone Calculations Values**

| Method | H (ft) | W (lbs) | $D_n$ (ft) |
|--------|--------|---------|------------|
| Fetch  | 2.56   | 210     | 1.25       |

Using this design stone diameter value the crest width, layer thickness, and toe scour depth were found through equations 9, 10, and 11

$$B = 3D_n \quad 9$$

where  $B$  is the crest width in feet

$$r = 2D_n \quad 10$$

where  $r$  is the layer thickness in feet

$$S = 2D_n \quad 11$$

where  $S$  is the toe scour depth in feet. The crest width, layer thickness, and toe scour depth calculated for both the breakwater and revetment are shown in Table 7. The second layer key dimensions were determined to be half of the main layers dimensions.

**Table 7: List of the Main Key Layer Dimensions**

| B (ft) | r (ft) | S (ft) |
|--------|--------|--------|
| 3.75   | 2.5    | 2.5    |

The crest elevation of the breakwater and revetment are dependent on the depth at which they will be placed. For the following list of equations leading to finding the design crest elevation for each individual design used a breakwater depth of 2.06 feet and a revetment depth of 0.06 feet. The first step in determining the crest elevation for both designs is determining the mean wave period,  $T_{om}$ , found using equation 12

$$T_{om} = 0.8T_p \quad 12$$

where  $T_p$  is the wave period of 2.09 seconds.

The wave period was used to find the surf-similarity parameter,  $\delta_{om}$  in equation

$$\delta_{om} = \frac{\tan(\alpha)}{\sqrt{\frac{2\pi H}{gT_{om}^2}}} \quad 13$$

where  $\tan(\alpha)$  is the design slope ratio of 0.5 and  $g$  is acceleration due to gravity. Since  $\delta_{om}$  is between 1.0 and 1.5, the 2% wave run up,  $R_{2\%}$  was found through equation 14

$$\frac{R_{2\%}}{H} = A\delta_{om} \quad 14$$

where  $A$  is a coefficient for impermeable rock slopes listed in Table VI-5-5 of the Coastal Engineering Manual (CEM, 2020). The results of the above equations are provided in Table 8.

**Table 8: Mean Wave Period, Surf-Similarity Parameter, and 2% Wave Run Up Calculations**

| $T_{om}$ (sec) | $L_{om}$ | $R_{2\%}$ (ft) |
|----------------|----------|----------------|
| 1.67           | 1.15     | 2.99           |

The 2% wave run up was added to the design water level in order to establish a design crest elevation dependent on significant wave height. The design water level for the breakwater included the depth of the structure of 2.06, the 50 year sea level rise design parameters of 0.625 feet, and the design parameter of a storm surge at 4.875 feet. Therefore, the design water level used for the breakwater was 9.33 feet. The revetment design water level was based off of the same sea level rise and storm surge requirements but only at a depth of 0.6 feet. Therefore, the design water level used for the revetment was 7.33 feet. The resulting wave crest elevations were rounded up to apply a built in factor of safety as shown in Table 9.

**Table 9: Crest Elevation for Each Respective Design**

| Design     | Crest Elevation (ft) |
|------------|----------------------|
| Breakwater | 10.5                 |
| Revetment  | 8.5                  |

The difference in heights due to the ocean gradient was considered. The complete dimensioned design sketches made in the online SketchUp program for the breakwater and revetment can be seen in Figure 41 and Figure 42, respectively. In both figures the orientation is based on the shoreline being to the left, the change in height is addressed at the center of the sketch shown below.

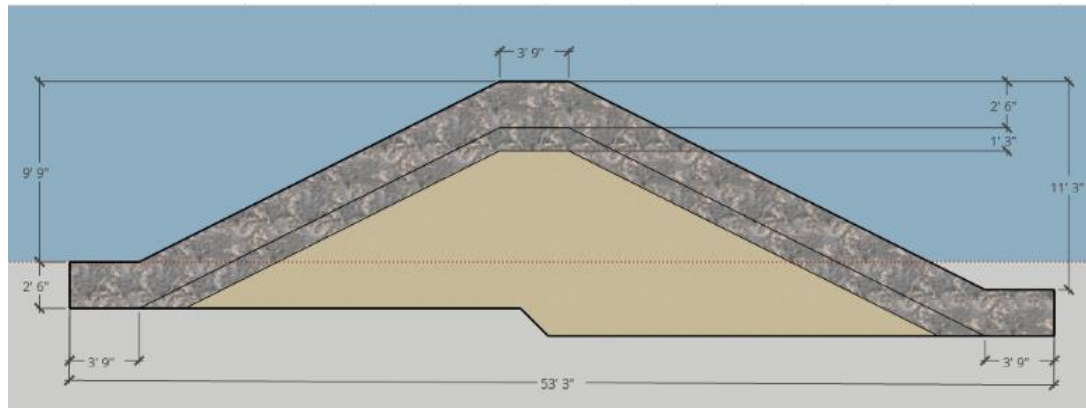


**Figure 41: Dimensioned Breakwater Design Sketch**

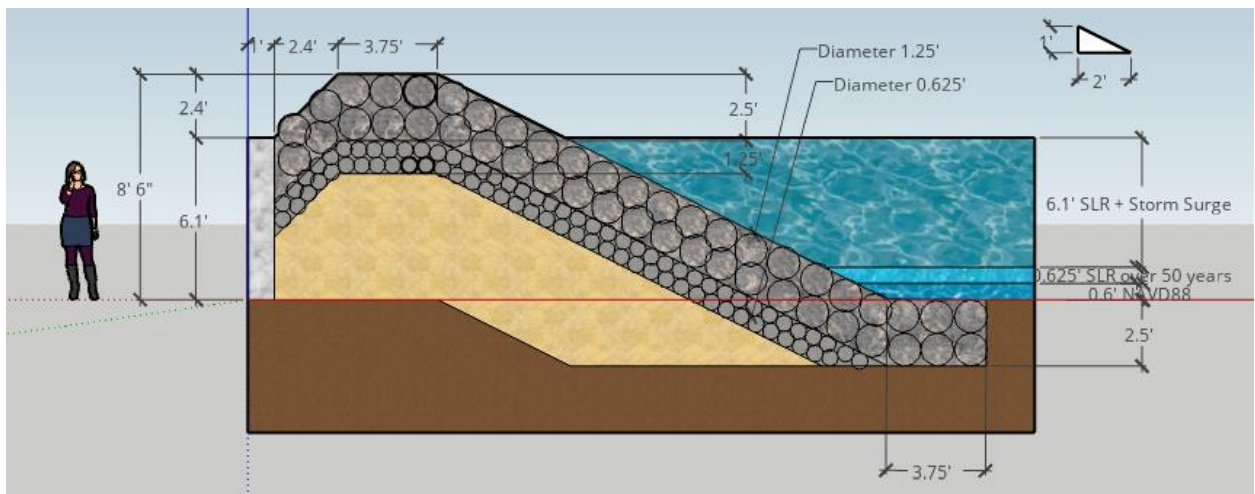


**Figure 42: Dimensioned Revetment Design Sketch**

#### ***4.2.2 Further Revetment and Breakwater Calculations***

After getting the base calculations, the length of the breakwater and number of breakwaters was selected. Two different restrictions must be put in place based on basic designs of breakwaters. These restrictions are shown by equation 15 and equation

$$L_s > .8 L_g \quad 15$$

$$T > 0 \quad 16$$

where  $L_s$  is the length of the breakwater,  $L_g$  is the space between the breakwaters, and  $T$  is the length of sand accreted along the landward side of the breakwater.  $T$  is found by using equation 17. The constraint placed by equation 15 is mandatory, while equation 16 only applies if there is desired accretion landward of the breakwater.

$$T = L_s - .52L_g \quad 17$$

The number of breakwaters was the base of the calculation. To make the design both aesthetically pleasing and to meet the above requirements, two breakwaters were selected. After several iterations of equations 15-17, 80 feet was the decided length. With an 80 feet length there would be a buildup of sediment behind the breakwaters, a large enough gap between the breakwaters and an ample distance from the northern property line. Using the above equations a distance from the extended northern property line was 20ft, and the separation between the two breakwaters was 80 feet.

Along with the number and length of breakwaters, the distance from the seawall to the breakwater was also selected. The design height of the breakwater was made from the crest elevation as stated before. The topography consisted of a one foot drop every 40 feet from the seawall. To match the crest elevation with the drop in depth, a distance of 80 feet was selected from seawall to the center of the breakwater as shown in Figure 43.

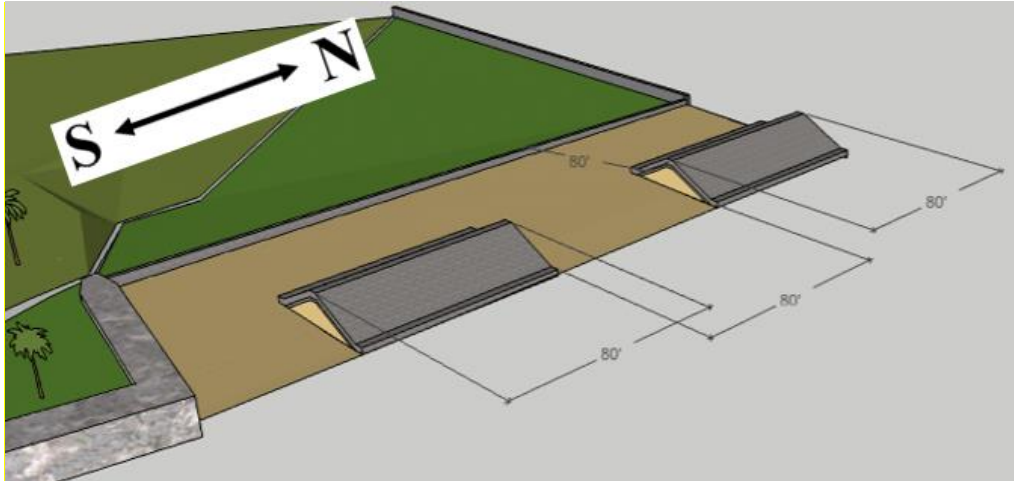


**Figure 43: Aerial View of Northern Seawall with Two Breakwaters**

The proposed design for the revetment would be located on the seaward side of the south seawall, as seen in Figure 44 and Figure 45. The revetment would reach along the existing seawall until it intersects with the larger main seawall to the North and the raised seawall to the South. Discussion of how the dimensions of this design were formed with regards to the design water level desired by the customer and further explanation is provided in the section above.

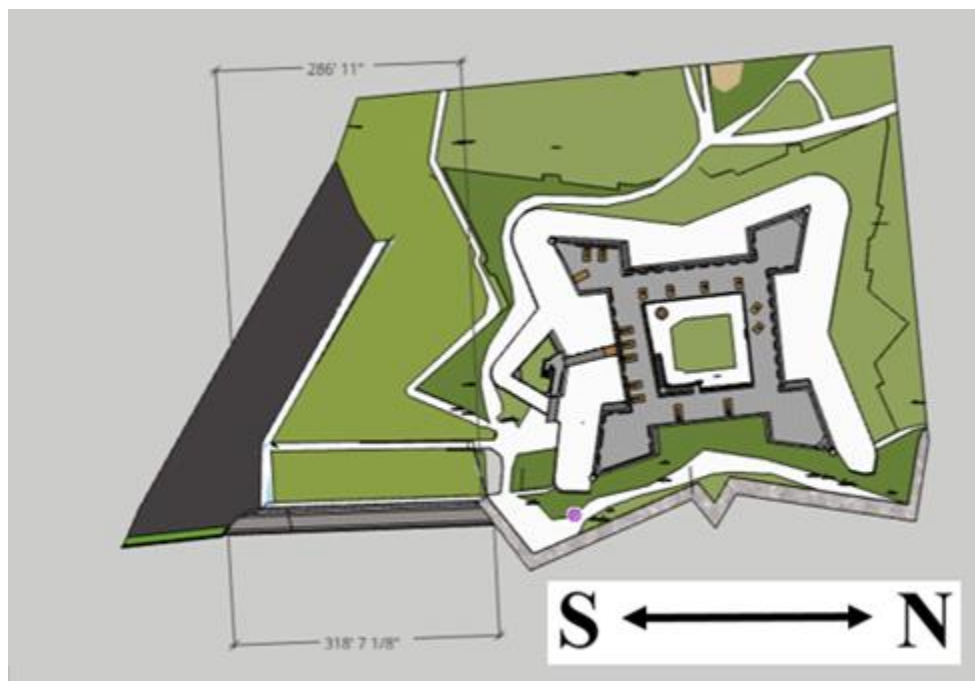


**Figure 44: Top view of proposed revetment design**

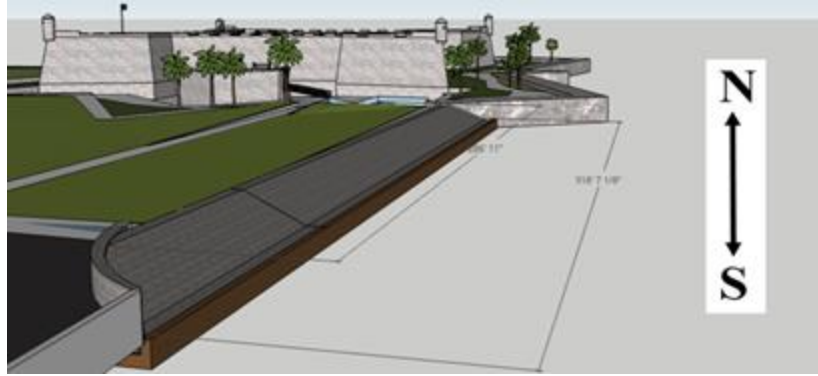


**Figure 45: Seawall view of proposed revetment design**

### ***4.3 Earthen Levee Design***

The proposed earthen levee stands at a height of 6 feet with a diameter of 12 feet, as shown in Figure 46. By placing the earthen levee 2.5 feet above NAVD88, the crest of the semicircle will reach 8.5 feet above NAVD88. At this height above NAVD88, the structure will provide flood protection with an added factor of safety with regards to the project's design water level of sea level rise and storm surge equating to 6.1 feet above NAVD88. The position of the earthen levee and where it intersects with the existing seawall is shown in Figure 46.

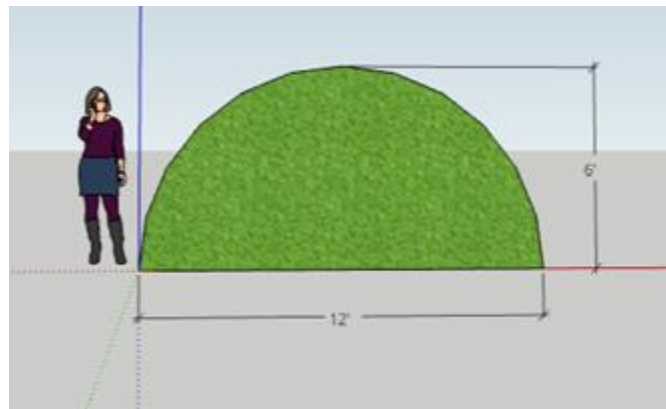


**Figure 46: Profile view of earthen levee design**

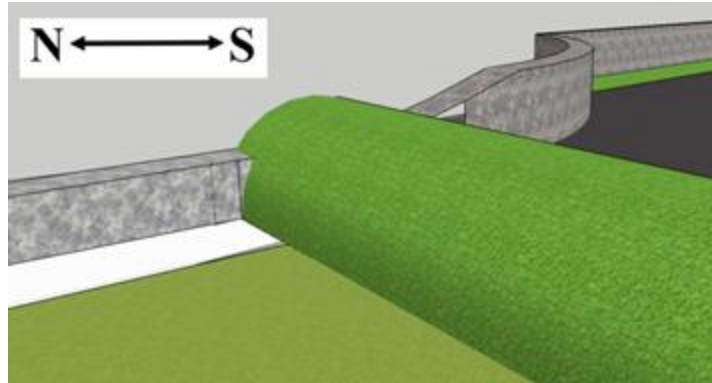


**Figure 47: Intersection of the proposed earthen levee to Seawall**

Construction of the earthen levee will mimic the design in Figure 48. Key aspects include a solid soil core with sod coverage that matches the natural surrounding landscape.

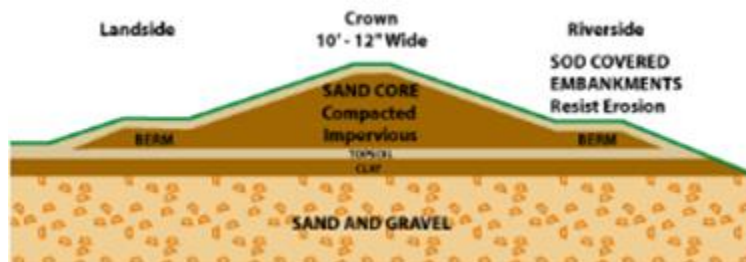


**Figure 48: Typical construction of an earthen levee (Earthen City Levee District, 2006)**

Figure 49 and Figure 50 show varying perspectives of the natural and aesthetically pleasing look of the design. These figures also illustrate how the earthen levee acts as a barrier around the northern edge of the parking lot, keeping the water in the open area between the parking lot and the northern hills coming from the fort. The earthen levee ends as it intersects with the western sidewalk on the crest of a downward

sloping hill, which the earthen levee will naturally merge with to complete an effective flood barrier.

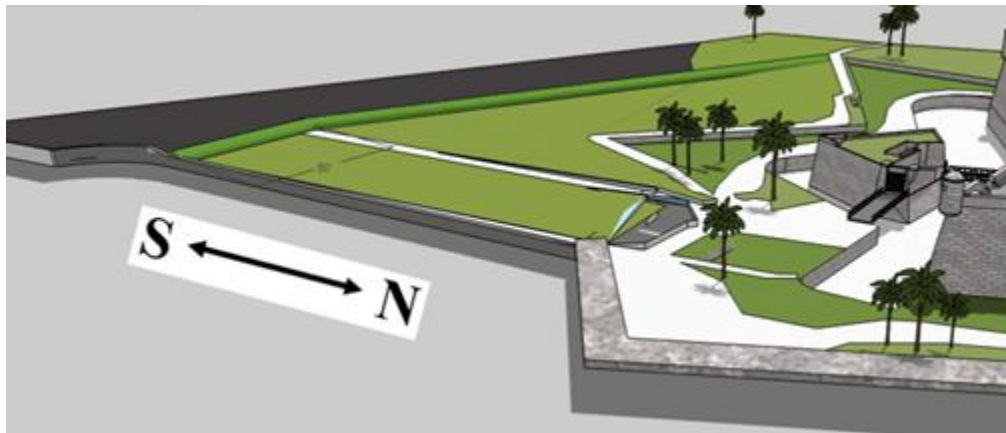


**Figure 49: SketchUp view of prospective earthen levee design**

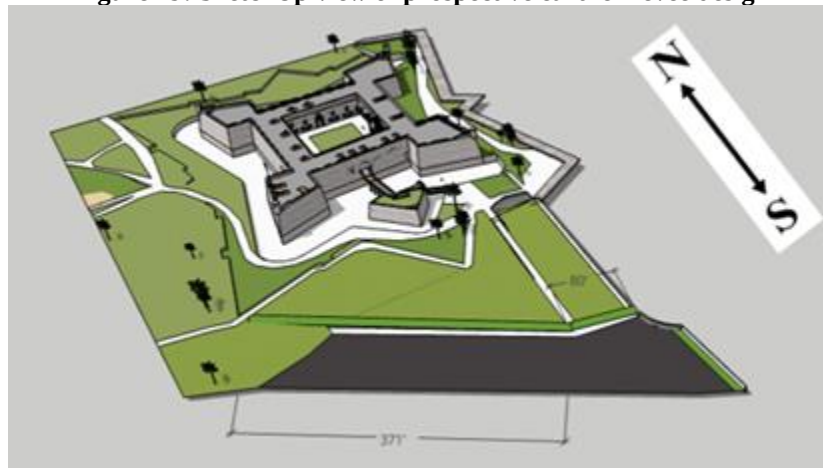


**Figure 50: Southern aerial view of the proposed earthen levee**

This design intersects two of the primary walkways apart of the fort, as seen in Figure 51. Public accessibility would be hindered without additional design considerations. Possible solutions include a ramp with a 3:1 slope or greater in order to accommodate visitors with wheelchair or other physical limitations, as shown in Figure 52 and Figure 53. This slope would encroach on the limited space of the parking lot potentially taking away parking spaces. Another proposed solution would be to incorporate a hinge flood gate or a slide in removable flood barrier, as shown in Figure

54. This design would require the customer to properly employ the barrier before any major storm event.

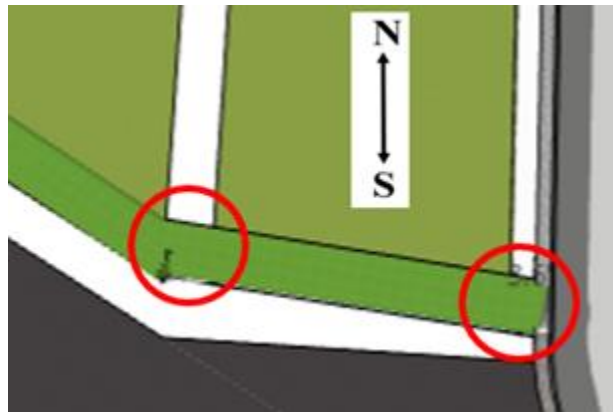


**Figure 51: Areas of limited public accessibility**

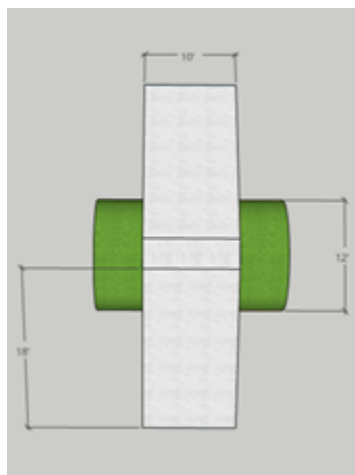


**Figure 52: Top view of proposed ramp solution**



**Figure 53: Side view of proposed ramp solution**

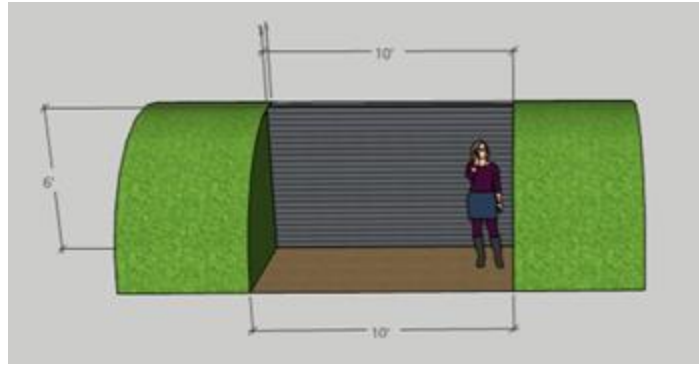


**Figure 54: Proposed gate in flood barrier solution**

Another limitation of the earthen levee design is the vulnerability of the east side of the parking lot, indicated in Figure 55 with a street view provided in Figure 56 and Figure 57. The seawall running north to south will not satisfy proper flood protection of the design water level considered in this report. If the customer intends to implement the earthen levee design, it is encouraged that they also look into and pursue an extension of the earthen levee along the eastern seawall of the parking lot.

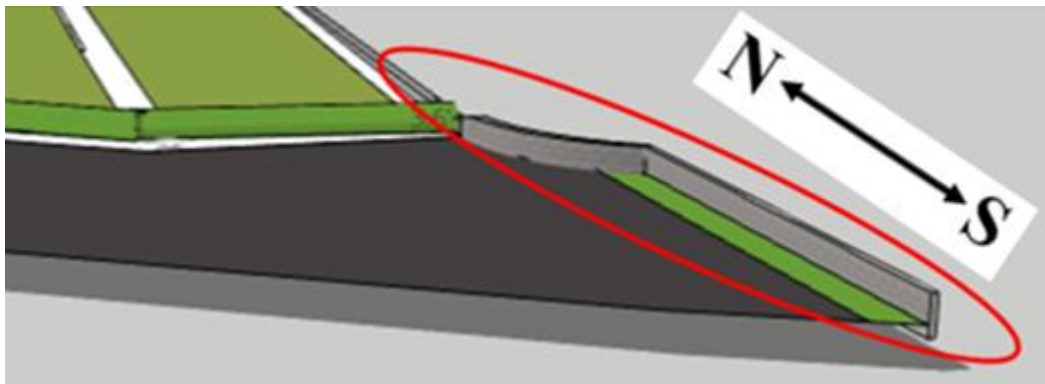


**Figure 55: Potential limitation of earthen levee design**



Figure 56: Street view of potential limitation of design (Google Earth, 2020)



Figure 57: Street view of the eastern seawall (Google, 2020)

#### 4.4 Beach Nourishment Calculations

The optimal design for the beach profile created through beach nourishment was developed in accordance with guidelines laid out in the Coastal Engineering Manual (CEM), and all equations used come from this source. To start, the equilibrium beach profile equations were used to determine the curvature of the profile from the depth of closure and working up until it reaches MSL. The underwater portion of the equilibrium beach profile is show by equation 18

$$h = Ay^{\frac{2}{3}} \quad 18$$

where  $h$  is the depth,  $y$  is the horizontal distance, and  $A$  is the A parameter. The A parameter is found using Table 10, which shows a CEM table that relates a sediment sample's  $D_{50}$  value to its A parameter given in units of  $m^{1/3}$ .

**Table 10: Recommended A Parameter Values Based on D<sub>50</sub> (Anonymous, 2012)**

| D(mm) | 0.00  | 0.01   | 0.02   | 0.03   | 0.04   | 0.05   | 0.06   | 0.07   | 0.08   | 0.09   |
|-------|-------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| 0.1   | 0.063 | 0.0672 | 0.0714 | 0.0756 | 0.0798 | 0.084  | 0.0872 | 0.0904 | 0.0936 | 0.0968 |
| 0.2   | 0.100 | 0.103  | 0.106  | 0.109  | 0.112  | 0.115  | 0.117  | 0.119  | 0.121  | 0.123  |
| 0.3   | 0.125 | 0.127  | 0.129  | 0.131  | 0.133  | 0.135  | 0.137  | 0.139  | 0.141  | 0.143  |
| 0.4   | 0.145 | 0.1466 | 0.1482 | 0.1498 | 0.1514 | 0.153  | 0.1546 | 0.1562 | 0.1578 | 0.1594 |
| 0.5   | 0.161 | 0.1622 | 0.1634 | 0.1646 | 0.1658 | 0.167  | 0.1682 | 0.1694 | 0.1706 | 0.1718 |
| 0.6   | 0.173 | 0.1742 | 0.1754 | 0.1766 | 0.1778 | 0.179  | 0.1802 | 0.1814 | 0.1826 | 0.1838 |
| 0.7   | 0.185 | 0.1859 | 0.1868 | 0.1877 | 0.1886 | 0.1895 | 0.1904 | 0.1913 | 0.1922 | 0.1931 |
| 0.8   | 0.194 | 0.1948 | 0.1956 | 0.1964 | 0.1972 | 0.198  | 0.1988 | 0.1996 | 0.2004 | 0.2012 |
| 0.9   | 0.202 | 0.2028 | 0.2036 | 0.2044 | 0.2052 | 0.206  | 0.2068 | 0.2076 | 0.2084 | 0.2092 |
| 1.0   | 0.210 | 0.2108 | 0.2116 | 0.2124 | 0.2132 | 0.2140 | 0.2148 | 0.2156 | 0.2164 | 0.2172 |

The design D<sub>50</sub> value used was 0.15 mm, the average of the two wet sediment samples taken from the site, giving an A Parameter of 0.084 m<sup>1/3</sup>. It was decided that the beach will be nourished out to the base of the breakwater, 80 feet from the seawall. This ensures that if the beach nourishment is used in conjunction with the breakwater, as recommended, that only the minimum amount of sediment is being used as to optimize the cost. Starting at the base of the breakwater, which is -0.5 feet NAVD88 according to the GIS bathymetry data, and working backwards, it was found that the beach profile will reach MSL at a distance of 26.2 feet from the breakwater.

Once the beach profile reaches MSL, it will be dictated as shown in equation 19

$$h = \frac{y}{m} \quad 19$$

where  $m$  is the design slope of the beach face. The beach profile will continue like this until reaching the beach berm, where it will flatten out and extend landward far enough to be effective in flood prevention even in a 50 year storm event. The erosion of the beach berm in a storm event,  $R_B$ , is governed by equation 20

$$R_B = S \frac{x_b - \frac{h_b}{m}}{B + h_b - \frac{S}{2}} \quad 20$$

where  $S$  is the storm surge,  $B$  is the height of the berm,  $h_b$  is the wave height at the new depth of closure, which is governed by equation 21, and  $x_b$  is the parameter governed by equation 22

$$h_b = 0.76H_{mo} \quad 21$$

$$x_b = \left(\frac{h_b}{A}\right)^{3/2} \quad 22$$

where  $H_{mo}$  is the significant wave height. The width of the beach berm,  $W$ , was found using equation 23

$$W = R_B + FS \quad 23$$

where  $FS$  is the factor of safety built into the width of the beach berm. The purpose of the factor of safety is to overdesign the beach berm in order to ensure that even after a 50 year storm, there will still be beach berm left over for continued flood prevention.

Finally, the volume of sand necessary to complete the beach fill,  $V_d$  was found by using simple area calculations of the beach profiles side view and subtracting the area that is already occupied naturally, then multiplying this area by the length of the beach.

To find the optimum design for the beach nourishment project, the process described above was iterated until the dimensions were found that offer the most flood protection while using the smallest volume of sand, making the design as cost effective as possible. For the calculations, a significant wave height of 2.56 feet was used, in accordance with the fetch driven wave height found previously. If the beach nourishment

is used along with the breakwater solution as recommended, this significant wave height will be an over estimation, as the breakwater will diminish wave activity experienced at the beach front. However, the overestimation is not drastic, and serves the design well by incorporating a built-in factor of safety. The storm surge used in calculations was 4.88 feet, in accordance with the 50 year design water level. The factor of safety used was 50 feet, which is intended to be enough in terms of flood prevention while not being excessive to avoid unnecessarily driving up cost. The beach nourishment calculations for the final design done in accordance with the process described above can be seen in Table 11.

**Table 11: Beach Nourishment Calculations**

|       |                           |
|-------|---------------------------|
| $m$   | 0.10                      |
| $B$   | 7.00 (ft)                 |
| $h_b$ | 1.95 (ft)                 |
| $x_b$ | 61.54 (ft)                |
| $R_B$ | 31.52 (ft)                |
| $W$   | 81.52 (ft)                |
| $V$   | 92,110 (ft <sup>3</sup> ) |

The table shows the height of the berm, this puts the design berm crest at +7.0 feet NAVD88, high enough to prevent any flooding on the north side. The design beach berm will extend 91.7 feet landward of the current seawall. The final design of the beach profile after beach nourishment, including both the area lost due to storm erosion in a 50 year storm and the existing ground and seawall already at the location, can be seen in Figure 58.

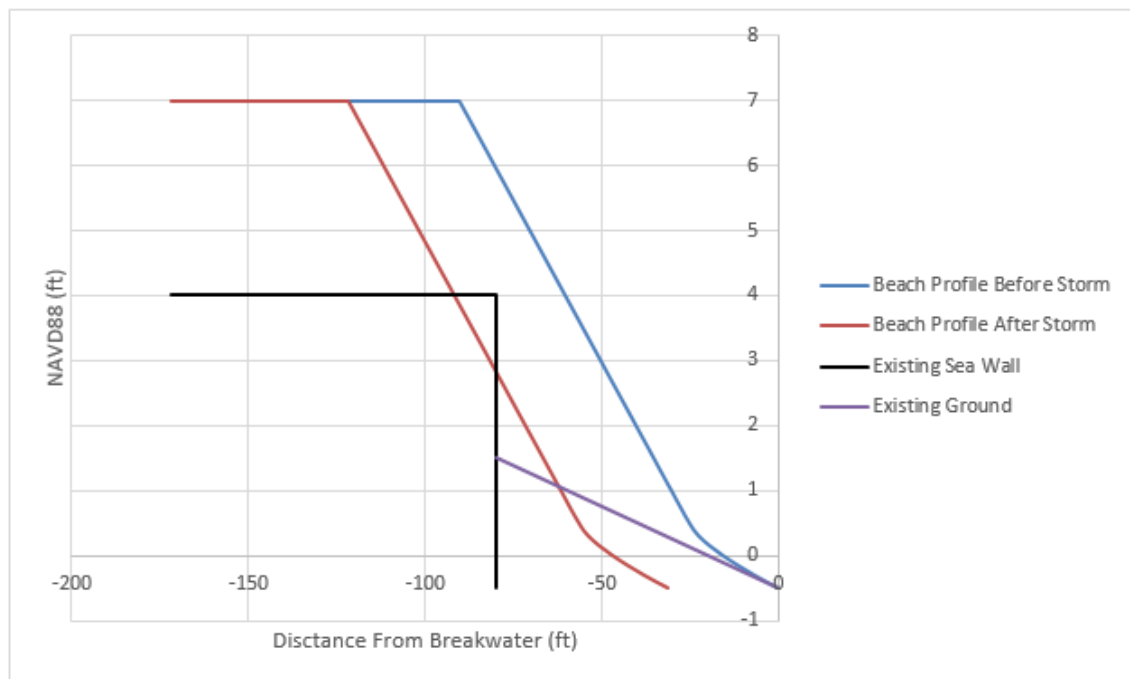


**Figure 58: Final Beach Profile Design**

## **Cost Estimation and Pricing**

The pricings made for the following considerations were based on both the cost data book “*RS Means*” and Florida’s Department of Transportation spreadsheet pricings. The unit cost for each material included both the material itself and the cost on installation. Between the four different design considerations, there were five different bid items: Armor Stone, Bedding Stone, Filter Fabric, Filling, and the Dune Planting. The following sections will go into more detail on the unit cost of each bid item and the individual cost of each consideration.

### ***5.1 North Design Considerations and Pricing***

This section will focus on the final design pricings for the northern area of Fort Castillo National Park. These designs are concentrated around providing protection to the headquarters against flooding giving the National Park association the ability to maintain operations during a flooding or storm event.

#### ***5.1.1 Breakwater***

The north breakwater was the largest of the four considerations. Table 12 shows the breakdown of the pricing. The quantity was calculated by estimating the volume of the material then converting to tons. The armor stone is the exterior, the bedding stone is the next layer of smaller stone, the fill is sand and the filter fabric is the separating material between the bedding stone and sand fill. The filter fabric was the only bid item that was not measured in tons. Because of the small thickness, the Filter fabric was measure in square yards squared rather than tons. Taking up an area of about 8,700 ft<sup>2</sup> of the northern seawall, the pricing for the breakwater came to about \$1.9 million.

**Table 12: Price Breakdown for the North Breakwater**

| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
|---------------------------|---------------------------|----------|-------|------------|-----------------|
| Armor Stone               | Armor Stone Quantities    |          |       |            |                 |
|                           | Weight of Armor Stone     | 11,370.0 | TON   | \$ 100.63  | \$ 1,144,163.10 |
|                           |                           |          |       | Subtotal   | \$ 1,144,163.10 |
|                           |                           |          |       | Rounded    | \$ 1,144,000.00 |
| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
| Bedding Stone             | Bedding Stone Quantities  |          |       |            |                 |
|                           | Weight of Bed Stone       | 4,664.0  | TON   | \$ 98.33   | \$ 458,611.12   |
|                           |                           |          |       | Subtotal   | \$ 458,611.12   |
|                           |                           |          |       | Rounded    | \$ 459,000.00   |
| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
| Filter Fabric - Revetment | Filter Fabric Quantities  |          |       |            |                 |
|                           | Square Yards of Revetment | 2,880.0  | SY    | \$ 4.00    | \$ 11,520.00    |
|                           |                           |          |       | Subtotal   | \$ 11,520.00    |
|                           |                           |          |       | Rounded    | \$ 12,000.00    |
| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
| Fill for Revetment        | Earthwork for Revetment   |          |       |            |                 |
|                           | Fill Volume               | 17,750.0 | TONS  | \$ 18.00   | \$ 319,500.00   |
|                           |                           |          |       | Subtotal   | \$ 319,500.00   |
|                           |                           |          |       | Rounded    | \$ 320,000.00   |
|                           |                           |          |       | Total Cost | \$ 1,935,000.00 |

### 5.1.2 Beach Nourishment

Landward of the breakwater would be the beach fill. The beach nourishment design was the simplest when it came to cost analysis. As stated before the cost for the fill itself and the installation cost were combined into the unit cost. The fill volume for the sand came to approximately 49,000 yd<sup>3</sup> which was estimated around \$879,000 as shown in Table 13.

**Table 13: Price Breakdown for Beach Nourishment**

| BID ITEM           | DESCRIPTION             | QUANTITY | UNITS | UNIT COST  | TOTAL COST    |
|--------------------|-------------------------|----------|-------|------------|---------------|
| Fill for Revetment | Earthwork for Revetment |          |       |            |               |
|                    | Fill Volume             | 48,813.0 | CY    |            |               |
|                    | Fill Volume             | 48,813.0 | CY    | \$ 18.00   | \$ 878,634.00 |
|                    |                         |          |       | Subtotal   | \$ 878,634.00 |
|                    |                         |          |       | Rounded    | \$ 879,000.00 |
|                    |                         |          |       | Total Cost | \$ 879,000.00 |

## 5.2 South Design Considerations and Pricing

This section will focus on the final design pricing for the southern area of Fort Castillo National Park. These designs are concentrated around trying to minimize flooding into the parking lot, which in turn would provide protection to the city south of the Fort.

### 5.2.1 Earthen Levee

With only two bid items, the fill and the dune planting, the earthen levee was the most cost effective consideration. It is cost effective because of the size of project and the amount of material. Measuring in acres, the amount of grassing was minimal only coming to about a fifth of an acre. Additionally the fill volume, composed of both sand and dirt, was measure to about 13,500 yards<sup>3</sup> which is approximately a quarter of the fill needed for the beach nourishment. Pricing can be seen in Table 14.

**Table 14: Price Breakdown for the Earthen Levee**

| BID ITEM           | DESCRIPTION             | QUANTITY | UNITS | UNIT COST   | TOTAL COST    |
|--------------------|-------------------------|----------|-------|-------------|---------------|
| Fill for Revetment | Earthwork for Revetment |          |       |             |               |
|                    | Fill VolumeVolume       | 13,520.0 | CY    |             |               |
|                    | Fill VolumeVolume       | 13,520.0 | CY    | \$ 18.00    | \$ 243,360.00 |
|                    |                         |          |       | Subtotal    | \$ 243,360.00 |
|                    |                         |          |       | Rounded     | \$ 243,000.00 |
|                    |                         |          |       |             |               |
|                    |                         |          |       |             |               |
| BID ITEM           | DESCRIPTION             | QUANTITY | UNITS | UNIT COST   | TOTAL COST    |
| Dune Planting      | Grassing                |          |       |             |               |
|                    | AC of Grassing          | 0.2      | AC    |             |               |
|                    | AC of Grassing          | 0.2      | AC    | \$ 3,500.00 | \$ 665.00     |
|                    |                         |          |       | Subtotal    | \$ 665.00     |
|                    |                         |          |       | Rounded     | \$ 1,000.00   |
|                    |                         |          |       |             |               |
|                    |                         |          |       |             |               |
|                    |                         |          |       | Cost Total  | \$ 244,000.00 |

### 5.2.2 Revetment

The south wall revetment held the second highest cost behind the breakwater. Covering 298 feet there were large amounts of material needed for completion. Between

the armor stone, bedding stone, filter fabric and fill, the price came out to approximately 1.7 million dollars shown in Table 15.

**Table 15: South Wall Revetment Pricing**

| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
|---------------------------|---------------------------|----------|-------|------------|-----------------|
| Armor Stone               | Armor Stone Quantities    |          |       |            |                 |
|                           | Weight of Armor Stone     | 9,800.0  | TON   | \$ 100.63  | \$ 986,174.00   |
|                           |                           |          |       | Subtotal   | \$ 986,174.00   |
|                           |                           |          |       | Rounded    | \$ 986,000.00   |
| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
| Bedding Stone             | Bedding Stone Quantities  |          |       |            |                 |
|                           | Weight of Bed Stone       | 4,270.0  | TON   | \$ 98.33   | \$ 419,869.10   |
|                           |                           |          |       | Subtotal   | \$ 419,869.10   |
|                           |                           |          |       | Rounded    | \$ 420,000.00   |
| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
| Filter Fabric - Revetment | Filter Fabric Quantities  |          |       |            |                 |
|                           | Square Yards of Revetment | 8,940.0  | SY    | \$ 4.00    | \$ 35,760.00    |
|                           |                           |          |       | Subtotal   | \$ 35,760.00    |
|                           |                           |          |       | Rounded    | \$ 36,000.00    |
| BID ITEM                  | DESCRIPTION               | QUANTITY | UNITS | UNIT COST  | TOTAL COST      |
| Fill for Revetment        | Earthwork for Revetment   |          |       |            |                 |
|                           | Fill Volume               | 14,700.0 | CY    | \$ 18.00   | \$ 264,600.00   |
|                           |                           |          |       | Subtotal   | \$ 264,600.00   |
|                           |                           |          |       | Rounded    | \$ 265,000.00   |
|                           |                           |          |       | Total Cost | \$ 1,707,000.00 |

### 5.3 Total Cost

As stated before, each consideration in the project can either be done individually or in a combination of the customer's choosing. Below in Table 16 would be the total cost of the project if the customer selected all of the design considerations.

**Table 16: Total Price Breakdown**

| ITEM                 | TOTAL COST      |
|----------------------|-----------------|
| North Breakwater     | \$ 1,935,000.00 |
| ITEM                 | TOTAL COST      |
| South Wall Revetment | \$ 1,707,000.00 |
| ITEM                 | TOTAL COST      |
| Beach Nourishment    | \$ 879,000.00   |
| ITEM                 | TOTAL COST      |
| Earthen Levee        | \$ 244,000.00   |
| Total Cost           | \$ 4,765,000.00 |

## Conclusion

The team visited Fort Castillo, collected data, and was given the goal to limit erosion, uphold the cultural integrity of the site, and control flooding over a 50 year project duration. Fort Castillo is one of the main tourist attractions in St. Augustine, FL and is a pillar of the culture and history of the area. The main problem areas were erosion on the North and South side, and flooding concerns on the South side and into the headquarters during storms and heavy rainfall. After some preliminary design considerations, the team decided on the following solutions: a revetment on the South side, breakwaters on the North side, beach nourishment along the coastline, and an earthen levee along the parking lot. The team completed calculations and created potential designs on SketchUp for each solution. The team worked with Taylor Engineering to create a cost analysis of the individual solutions. The National Park Service will survey the solutions created and decide which are best suited for their needs and budget.

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