

Department of Naval Architecture and Ocean Engineering
Ocean Engineering Capstone Design

	Aquatic Park Pier (Muni Pier) Replacement Project by  (Team EOE) April 30, 2020
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Executive Summary

The National Park Service's (NPS) Aquatic Park Historic District is a National Historic Landmark. The park is composed of a beach, municipal pier, cove, and maritime museum. The entire district is located within Fisherman's Wharf on the Northern coast of San Francisco directly south from Alcatraz Island. The Municipal Pier (Muni Pier) provides coastal protection for the Aquatic Park Historic District. The pier extends out from Black Point with a curved radius. Underneath its deck, wave baffles attenuate waves in order to protect the coast. Muni Pier is ninety years old and in need of immediate replacement due to corrosion, weathering, and general material degradation. Currently, half of the pier's width is fenced off to visitors due to the damaged condition of the concrete safety railing. Tourists and residents alike can no longer enjoy some of the outstanding views from the pier due to safety concerns. A group of students from the Naval Academy has been tasked by the NPS to analyze the current pier and design a replacement. The group, The Pier Builders, consists of First Class Midshipman Elvin, Myers, Ostrowski, Prince and Russell. The team can be seen in Figure 1 below. Their technical advisor is Professor Kriebel, Ph.D., P.E., of the United States Naval Academy Ocean Engineering faculty.

The NPS has made it clear that any replacement design must have the same length and shape; however, the replacement may incorporate a different height, width, and wave attenuation system. The pier's historic and cultural significance as a landmark are of great importance to the NPS and need to be adhered to for any new design. The new breakwater on the pier is designed to optimize the reduction of wave energy coming into the cove from the San Francisco Bay. New pier designs incorporate a modern breakwater system and are designed to last at least 50 years. Modern concrete, corrosion, earthquake, and pile design are also used in the replacement. To minimize replacement cost, the alternative pier is designed slimmer than the current 60-foot wide pier. However, the design is also wide enough to accommodate an Emergency Medical Services (EMS) ambulance in the case of an incident occurring on the pier. This new width allows for an EMS to complete a three-point turn at any location on the pier, rather than having to drive all the way down to the circular end and turn around. The height of the replacement pier is chosen based on sea-level rise projections in San Francisco for the next fifty to one hundred years. Placement

of the new breakwater accounts for sea level rise as well as the need to attenuate waves from mean lower low water (MLLW) up to mean higher high water (MHHW). The structural calculations for the replacement pier will have to meet building codes as laid out by the Port of San Francisco (Port of San Francisco Engineering Division, 2019). Specifically, the piles, pile caps, and decking will have to be able to sustain live and dead loads at a factor of safety of 1.2 and 1.6.

Laboratory testing will be a vital part of selecting the replacement design. The test was done using the current breakwater in a 1:24 scale model in an attempt to most accurately represent the waves that the pier experiences in real life as well as keeping the model relatively small.

A scale model was fabricated and tested in order to accurately represent the current environment as well as simulate the wave forces acting on the San Francisco Aquatic Park Pier. Testing this model yielded the breakwater's ability to effectively attenuate waves, but not during intense wave conditions. A second model was constructed in order to resemble the alternative design that would be utilized in real time. From this model, extensive structural calculations were conducted to verify the validity of the thought process behind the proposed alternative design. Future plans for the implementation and real construction of the pier were discussed.



Figure 1: The Pier Builders in Aquatic Park with Muni Pier in the background (MIDN Myers, 2020)

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Introduction

The National Park Service needs a new Municipal Pier, due to the old structure deteriorating at a rapid pace, concrete spalling, and major sections of the pier being closed off. The new pier will maintain the shape and length of the old pier, but the height and width of the new pier will be varied in order to accommodate sea level rise, for the next fifty to one hundred years, and provide an advantageous marine environment. In addition, focus will be put on keeping the pier's present cultural and historical significance to the San Francisco Maritime National Historic Park. The pier is an iconic landmark of the San Francisco waterfront, and its wave baffle system is essential for protection of the Aquatic Park Cove. The new proposed pier design has been made with the purpose of acting as a breakwater to allow citizens to safely partake in recreational activities and tourism. In addition to this, the pier will also be used to protect historic ships that are docked in the harbor. The new breakwater will be designed in order to optimize the reduction of wave energy coming into the cove, improve reparability, and remain effective with future sea-level rise.



Figure 2: Location of the Municipal Pier in San Francisco (NPS Value Analysis Study, 2009)

Background

The Municipal Pier is located in San Francisco, California. The pier was erected in 1929, in order to provide protection for the recreational basin in the San Francisco area as shown in Figure 2. The pier is located in the heart of the San Francisco Waterfront and offers paramount views of the city skyline as well as the Golden Gate Bridge and Alcatraz Island. On the pier and within the cove tourists are able to fish, anchor small boats in the lagoon, swim or sail in the basin, and enjoy the exceptional views. Additionally, the pier also provides protection for the city sea wall and other national landmarks such as the historic ships of Hyde Street Pier. The Aquatic Park serves as a historical center for the city's 19th century merchant roots with restored historic vessels on station and the Maritime Museum located within (San Francisco Maritime National Historical Park, 2019). The NPS maintains and cares for all of the assets within the aquatic park, to include the Municipal Pier. The NPS is committed to replacing the current aged pier while maintaining its iconic location and scenic views.

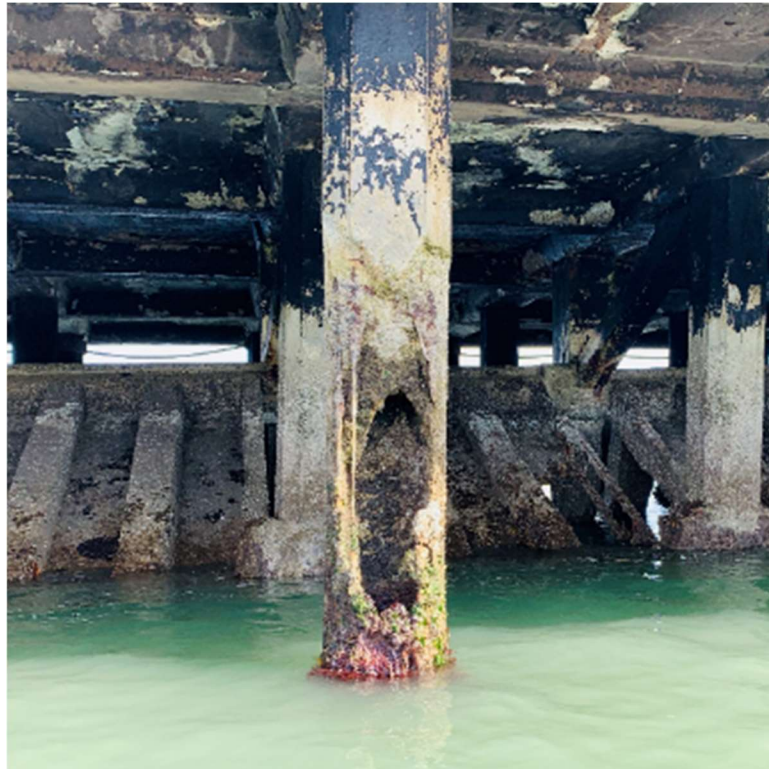


Figure 3: Concrete corrosion on pile and wave baffle (MIDN Connor Prince, Site Visit 2020)

The bulk of pier research was conducted using a variety of reports that were provided by the

NPS. These reports included the following: structural assessment checkup (GHD, 2016), muni-pier class-c cost estimate (Davido Consulting Inc, 2018), Fisherman's Wharf hydrodynamic circulation study (Phillip and Williams and Associates, 2004), pier corrosion study (Winzler and Kelly, 2008), and underwater pile survey (Winzler and Kelly, 2008). Selected pertinent information and multiple images have been used and referenced throughout this report. The main contributor for deterioration of the nearly 100-year-old pier is the severe concrete spalling caused by swelling highly corroded steel rebar as seen in Figure 3, above, and Figure 4, below.

Additionally, a site visit was conducted in February of 2020, to meet with the NPS and study the pier as it stands today. During this visit, the deterioration of the pier was obvious. Various segments of hand railing were missing and were replaced by wood paneling. Segments of the deck were sagging, due to years of damage from waves as well as from boating accidents. Visitor safety is paramount to the NPS and as such, the park had to close off half of the pier's deck with fencing in order to protect visitors from failed concrete handrails. Shown below in Figure 4 are up to date images documenting the deterioration of the pier's pedestrian deck and handrails. If no repairs or replacement occurs for the pier, the NPS will eventually have to close it off from visitation. During the site visit, a boat ride was taken with the NPS in order to view and analyze the damage to the underside of the decking. Wave baffle segments and piles have disappeared due to years of wear and tear from wave action and currents as shown in Figure 3. Due to the intricate and aged layout of the piles and baffles under the decking, repairing these components of the pier would be extremely costly and it is frankly out of the question. If the pier's baffles continue to fail, the protection provided to the Aquatic Park will continue to decrease. Located in Appendix C, are numerous up to date photos of the pier taken during the site visit.



Figure 4: Spalling due to rebar corrosion (MIDN Connor Prince, Site Visit, 2020)

The current pier is 60 feet in width, the total length of the pier is about 1400 feet and the height of the deck is approximately 12 feet above Mean Lower Low Water (MLLW) (NPS Value Analysis Study, 2009). There are 634 piles total, which are composed of 16-inch diameter timber piles with 22-inch squared concrete jackets. In total, there are 116 bents that consist of 5 vertical piles and two battered piles per bent. The vertical piles were all spaced at 12.5 feet on center and the bents are spaced at approximately 12 feet on center (GHD, 2016). At the edge of the pier deck there is a 5-foot overhang. The current design of the pier can be seen in Figure 5 below, and the overhead view of the current pier can be seen in Figure 6. The angle of the bents is unknown and requires further investigation. One of the largest concerns is that the concrete has begun to erode so that most, if not all of these piles will need to be replaced within 5-10 years. This is due to their resistance to dead and live loads will be essentially zero as well as resistance to seismic activity (NPS Value Analysis, 2008). The final design will be built to last for a minimum of 50 years.

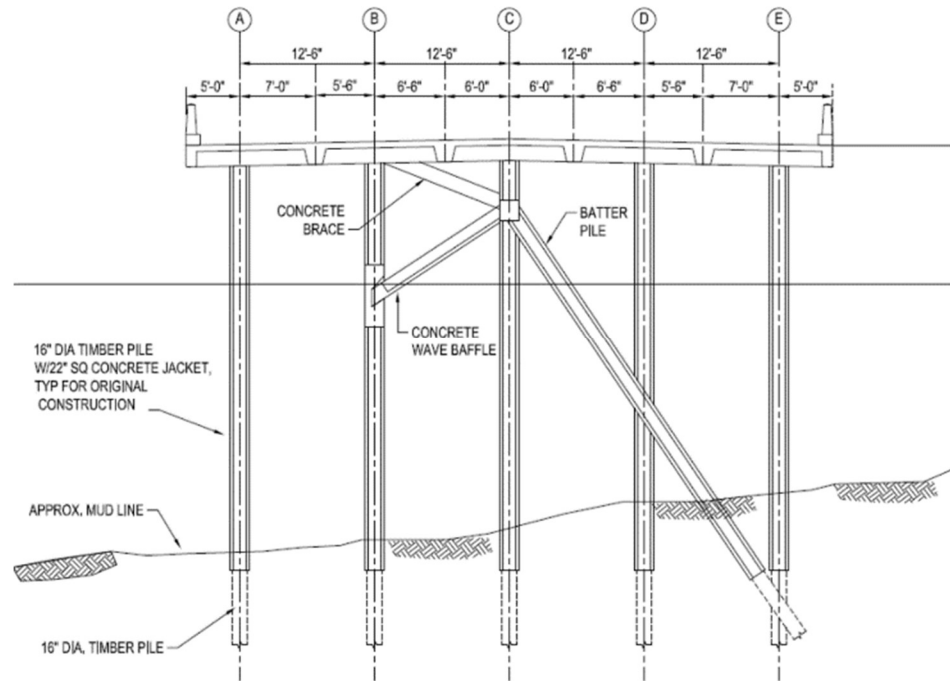


Figure 5: Current Pier Dimensions (NPS Value Analysis Study, 2009)

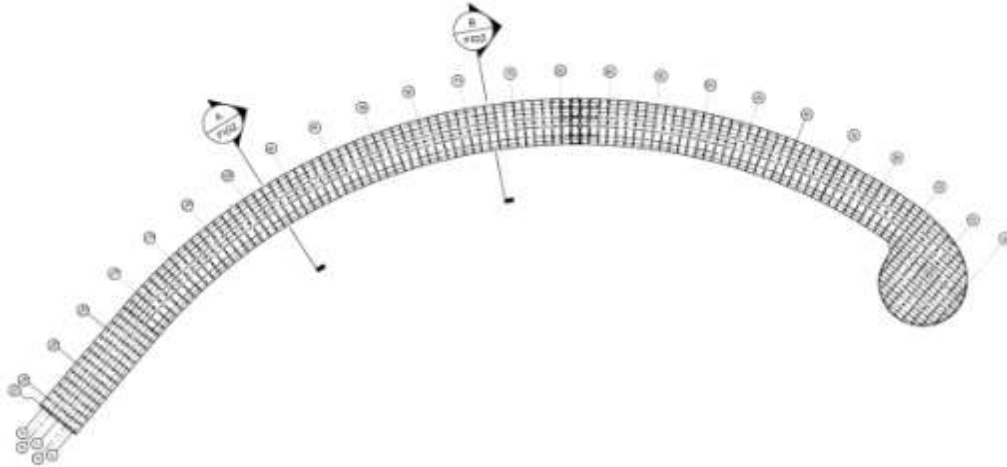


Figure 6: Pier Bird's Eye View Layout Plan (NPS Value Analysis Study, 2009)

Environmental Parameters

Due to being located on the mouth of the San Francisco Bay and just east of the San Andreas Fault, the San Francisco Pier and the area surrounding it is affected by several environmental factors, to include, tides, earthquakes, wind, waves, sea level rise, and bathymetry. As tides change, they move water in and out of the Bay, which causes moving water to interact with the breakwater. The San Andreas Fault brings seismic activity that moves the structure. To deal with this problem, a ductile design must be used in order for the pier to be able to bend during an earthquake without breaking. Winds blowing from the north create waves that crash against the system. The pier should be designed higher than the expected wave crests in order to ensure that the waves are not impacting the pier deck. Sea level must be taken into consideration when using a 50-year design life for the pier. With rising water levels all around the globe, the structure needs to be designed so that fifty years from now the sea level does not rise above the deck of the pier. Lastly, the bathymetry must be observed for when the piles are being placed into the water. These parameters will be further explained below.

Tides

San Francisco experiences semidiurnal tides, meaning that there are two high tides and two low tides every day.(Tidal Patterns, 2012) Transitioning from ebb to flood, water enters the San Francisco Bay and moves across the pier from left to right. When approaching ebb tide, water exits the Bay and flows from right to left. The ebb and flood graphs of the current pier depicting these velocities can be seen in Figures 8 and 9 below. In both figures, the fastest current velocities are red and orange, while the stagnant water is purple. The flood graph in Figure 8 shows the water coming into the Bay while the ebb graph in Figure 9 shows the water exiting the Bay. Data for the water levels was taken from the National Oceanic and Atmospheric Administration (NOAA) buoy 9414290 (NOAA Tides and Currents, 2019) the average water levels in relation to MLLW can be seen in the Table 1 below. The NOAA buoy is located west of Muni Pier, but just inside of the Golden Gate Bridge as seen in Figure 7. Therefore, it is able to get the measurement of tides and currents before they approach Muni Pier.



Figure 7: Buoy 9414290 location in San Francisco (Google Earth Pro, 2020)

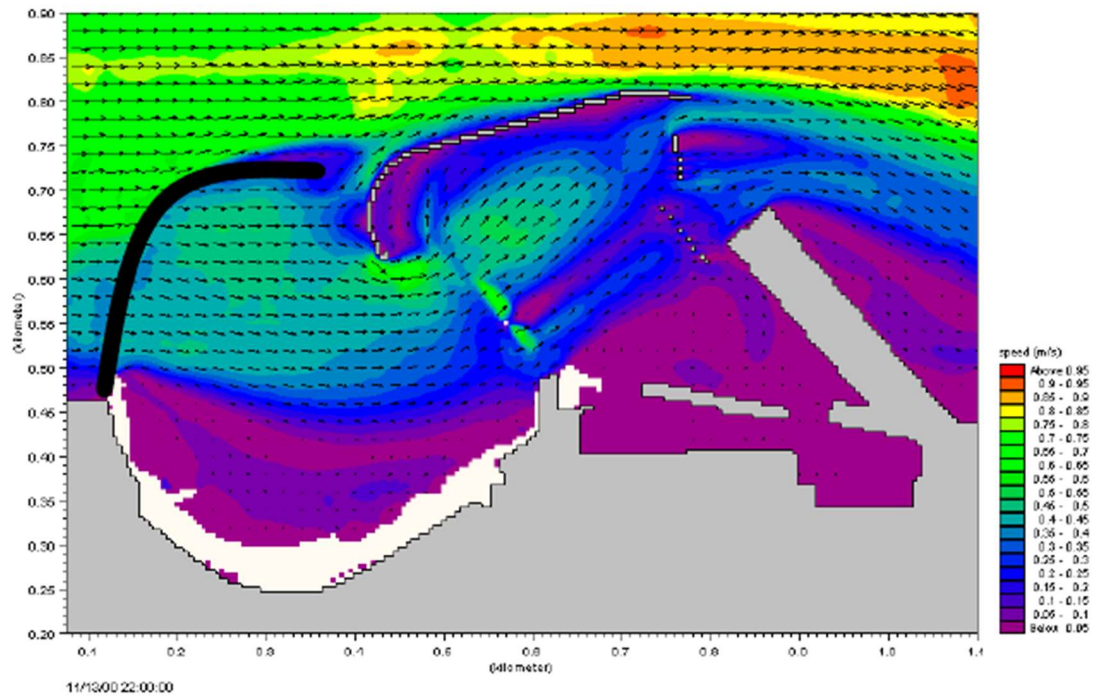


Figure 8: Typical Flood Flow Patterns and Velocities (Phillip Williams Association, 2004)

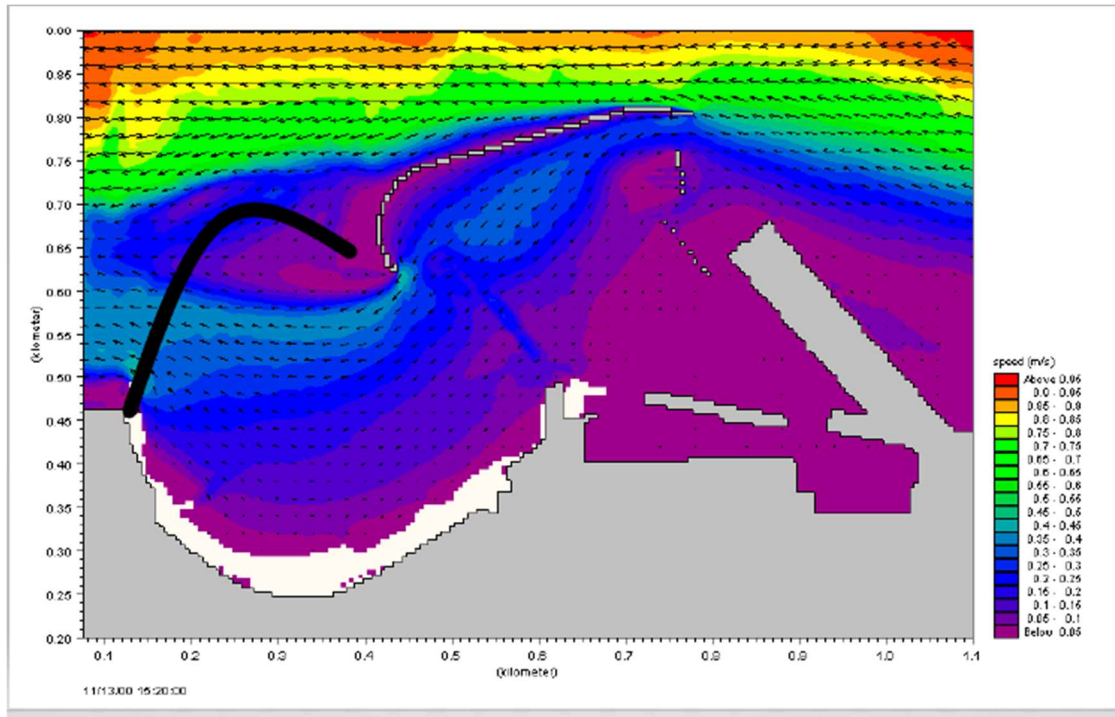


Figure 9: Typical Ebb Flow Patterns and Velocities (Phillip Williams Association, Fisherman’s Wharf Hydrodynamic Circulation Study, 2004)

Table 1: Water Level in relation to MLLW (NOAA Tides and Currents, 2019)

Tide Type	Water Level above MLLW (ft)
Mean Higher High	5.84
Mean Sea Level	3.12
North American Vertical Datum of 1988	-0.06
Max Tide	8.66

The max tide from MLLW is 8.66 ft. and the deck of the pier is only 12 ft. above MLLW, thus only giving four feet of clearance from the water. The values in addition to the figure help show how close the water could interact with the deck. With the current wave heights, which are discussed later on, water will flow over the top of the deck at max tide. The tides in reference to the pier can be seen in Figure 10 below.

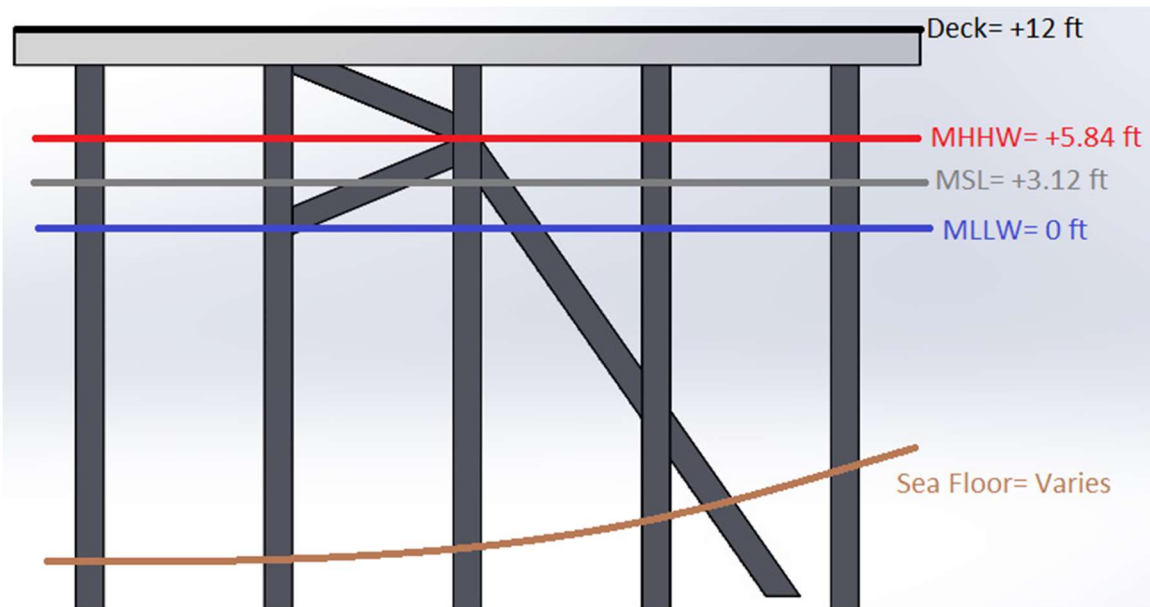


Figure 10: Current Pier with Datums (MIDN Connor Prince, SolidWorks, 2019)

Earthquakes

Due to the San Andreas Fault, this location is at risk of being hit by an earthquake. The fault is located off the west side of the Municipal Pier seen in Figure 11. On the west side of the fault, the plates move to the northwest, while on the east side of the fault the plates move to the southeast direction (Lynch, 2006). In the past 100 years, the largest earthquake that hit this area had a magnitude of 6.9. The 6.9 level earthquake knocked down the San Francisco-Oakland Bay Bridge. In total, this earthquake cost about 5 million dollars to repair all the damages. The damages aside from the bridge mainly included neighborhood housing and other smaller buildings. This earthquake occurred in 1989. (San Francisco Earthquake of 1989, 2009) It is important to keep in mind that the design for the pier should be ductile as opposed to a rigid design. Possible designs include steel cables, fuses, and concrete technology (Bergeron, 2009). The steel cables would be used to tie the joints together. The cables would stretch in the case of an earthquake but not break. This is a good option, but the problem with the cables is that they are more expensive than the other options. The fuses are the second option and in the case of an earthquake, they would break in a controlled manner. This is good because the pier would not completely fall apart. The fuses provide safety for the people on the pier, and they also can be replaced easily once they have broken. The last design option is the concrete technology. In this

possibility, the concrete pilings would be composed of short steel fibers that would heat up by using the energy of the earthquake. This technology would result in less breaks in the pier. After reviewing all the design possibilities, it was determined that a combination of the designs, such as steel cables and concrete technology, would be best (Bereron, 2009).



Figure 11: San Andreas Fault (Lynch, 2019)

Wind-Wave

Despite the majority of wave action at the mouth of the bay originating from west of the structure, the pier and breakwater experience minimal interaction due to the waves diffracting as they enter the bay. Additionally, the wind blowing in from the Pacific Ocean reaches land before hitting the pier, thus the structure experiences little wind action. The analysis of the direction of the waves that interact with the pier was based on conversations with and recommendations from the customer as well as data collected from the sight visit. Based on the customers recommendation, the waves created by wind perpendicular to the pier were analyzed. Using the U.S. Army Corps of Engineers' wind-wave forecasting method, two fetches were measured, one to the northwest (Figure 12) and one to the northeast (Figure 13). The wind speeds were found using the American Society of Civil Engineers (ASCE) wind map (ATC Council Hazards by Location, 2019) and the fetch distances were calculated using the ruler function on Google Earth Pro. With a 3-second wind speed of 74 miles per hour for a 50-year return period, the analysis for the fetch to the northwest yielded a significant wave height of 1.28 meters and a wave period of 3.11 seconds. Additionally, the analysis for the fetch to the northeast resulted in a significant

wave height of 1.29 meters and a wave period of 3.12 seconds.

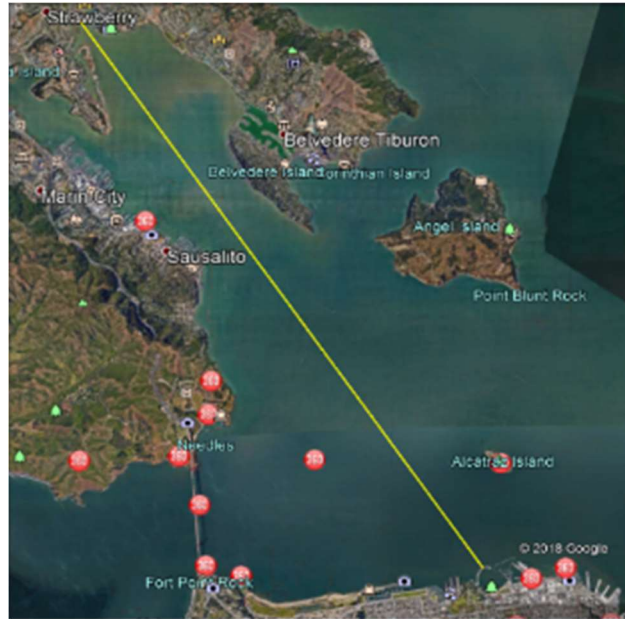


Figure 12: Northwest Fetch, 11615.31 meters (Google Earth Pro, 2019)

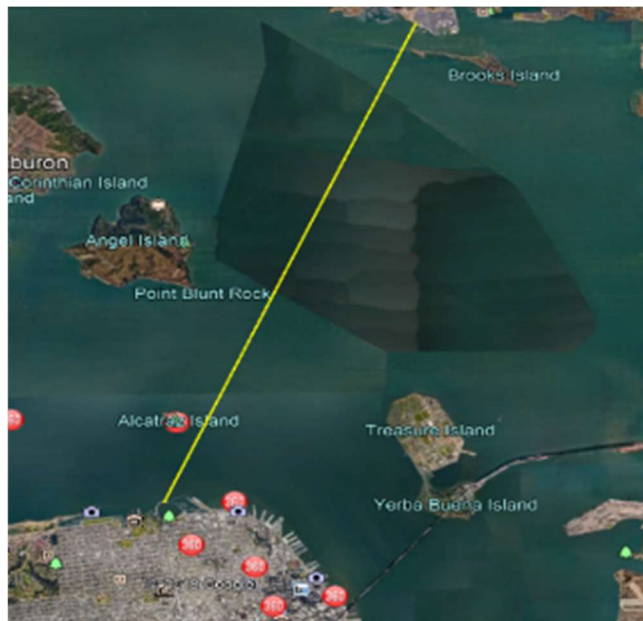


Figure 13: Northeast Fetch, 11738.21 meters (Google Earth Pro, 2019)

After converting the wind speed to meters per second, Figure 14 and Equation 1 were used to find the ratio of the 3-second wind speed versus the 1-hour wind speed. The following figure and three equations were taken from the Naval Academy Coastal Engineering textbook *Coastal*

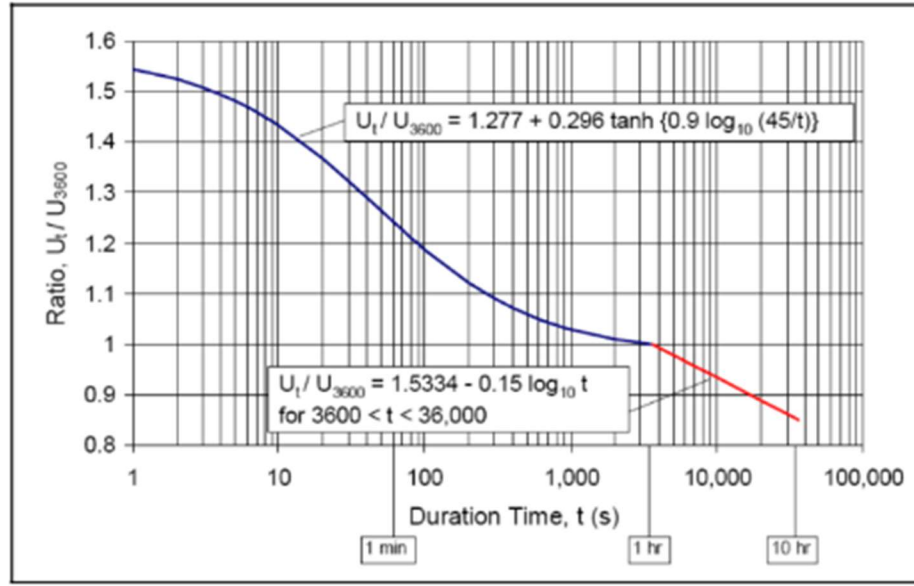


Figure 14: USACE ratio of wind speed of any duration U_t to the 1-hr wind speed U_{3600} (Kriebel, 2005)

$$\frac{U_t}{U_{36}} = 1.277 + 0.296 \tanh \left\{ 0.9 \log_{10} \left(\frac{45}{t} \right) \right\} \quad (1)$$

Where U_t is the wind speed for the time value initially found in m/s, U_{3600} is the 1-hr wind speed in m/s, and t is the initial time value in seconds. After the ratio was found, the 1-hr wind speed was solved by dividing the ratio from U_t . Following the rest of the process (Appendix A), the significant wave height and peak wave period in Equations 2 and 3, respectively.

$$H_s = \left(\frac{u_*^2}{g} \right) * 0.0413 * \sqrt{\frac{g * X}{u_*^2}} \quad (2)$$

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

Where H_s is the significant wave height in meters, u_* is the friction velocity in m/s, g is the gravitational constant 9.81 m/s^2 , X is the fetch in meters, and T_p is the peak wave period in seconds. This process was completed twice, once for the northwest fetch and the second time for the northeast fetch.

Sea-Level Rise

As time progresses, average water levels continue to rise and in order for the new pier to survive for fifty years, these values and trends were analyzed. USACE projects a 15-inch rise in water level by 2050 in California (Figure 15); however, NOAA states that sea level rise is occurring at 1.96 mm per year at Buoy 9414290. At the previously mentioned rate, by 2050 the water will rise by 3.86 inches (Figure 16). Additionally the trend line in Figure 16 suggests that sea level rise will accelerate in the coming years (NOAA, 2019).

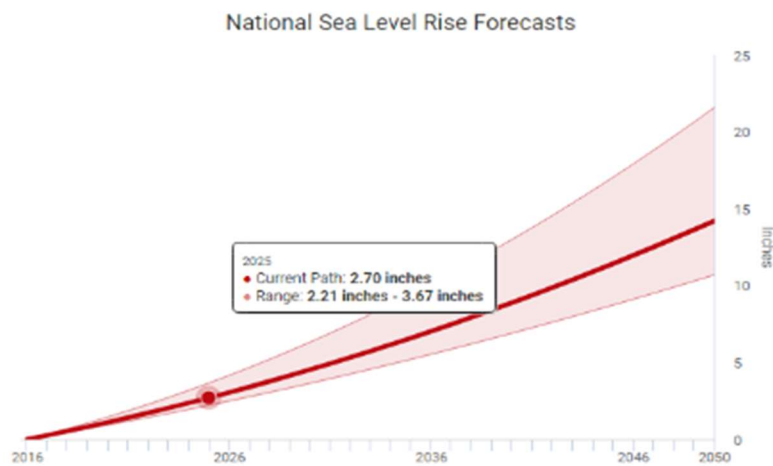


Figure 15: USACE Sea Level Rise Forecast (The Future of Sea Level Rise, 2019)

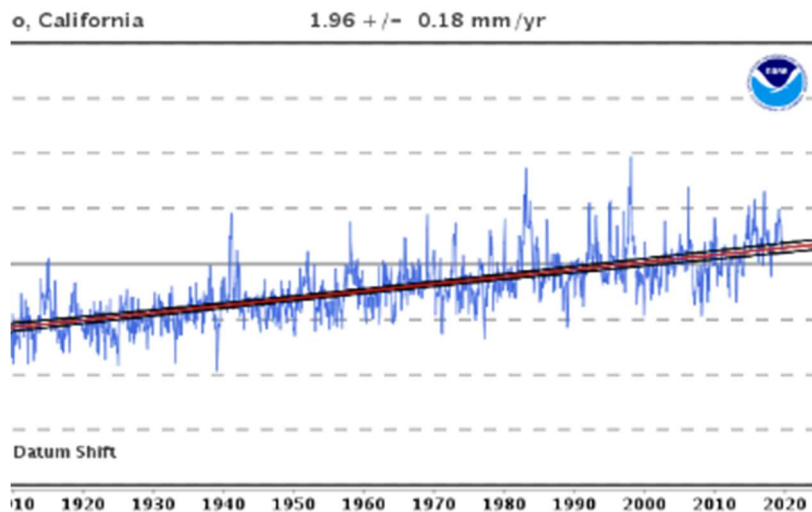


Figure 16: NOAA Sea Level Rise San Francisco (NOAA Meantrend.png, 2019)

Independent of these federal agencies, in 2018, the state of California released updated sea-level rise guidance. Catalyzed by a report written by the California Ocean Protection Council's Science Advisory Team (OPC-SAT), 2013 projections were updated to account for rapid advances in sea-level rise science. Using modern methods, which account for extreme polar melting scenarios, these projections were created by the OPC-SAT. The projections stated that if emissions remain high and polar melting continues to accelerate, San Francisco has a 66% probability of experiencing 1.6-3.4 ft. of sea-level rise by 2100 (State of California Sea-Level Rise Guidance, 2018). The 3.4 ft. sea-level rise assumes a low risk aversion scenario in accordance with the pier. The criteria for the low risk aversion is described within Appendix B in Table B2. This was considered for the replacement pier in order to be conservative.

Bathymetry

The aerial view of the pier in Figure 2 shows the shoreline in the bottom of the picture and the San Francisco Bay in the top of the picture. In a similar layout, Figure 17 depicts the bathymetry of the area surrounding the pier. The three dimensional coordinates provided by the NPS and matrix laboratory (MATLAB) were utilized to plot them. At the bottom of the graph is the shoreline with shallow depths and at the top is the start of the bay. The lagoon has a shallow slope when moving toward the pier, whose edges are outlined by the two black lines in the middle. The slope increases in steepness dramatically at and under the pier reaching depths of forty-four feet.

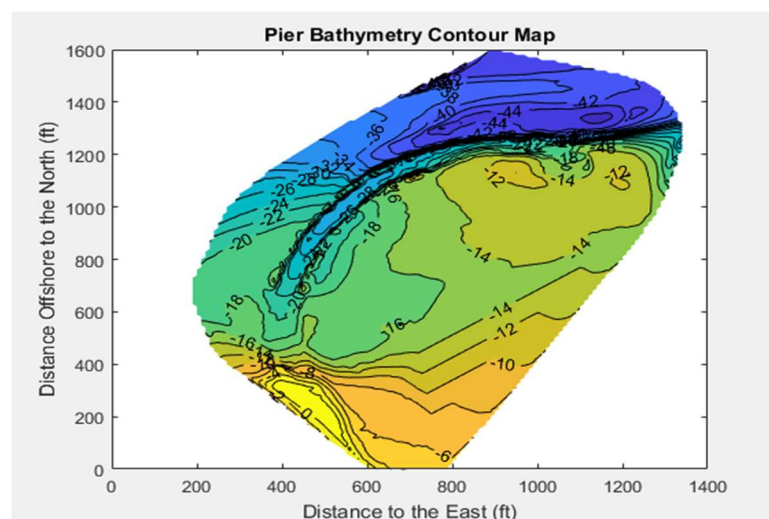


Figure 17: Bathymetry of Recreational Area (NPS Geodetic Data, 2019)

Current Design

In the current design, Figure 5, a wave baffle system below the decking is in place to attenuate waves that are coming into the Aquatic Park Historic District from the San Francisco Bay. These waves come in from the northeast and northwest of the pier, as explained in the wind-wave analysis. Figure 10 shows a cross section of the pier in addition to how it interacts with a variety of water levels based on the tides. Below, Figure 18 shows a SolidWorks isometric view of the current design with a translucent deck. The baffle is located in the middle of the deck and has a series of batter piles, which support the decking. Rows of five piles, otherwise known as bents, hold up the decking and the baffle system. The dimensions of the piles, bents, and decking are shown in Figure 5.

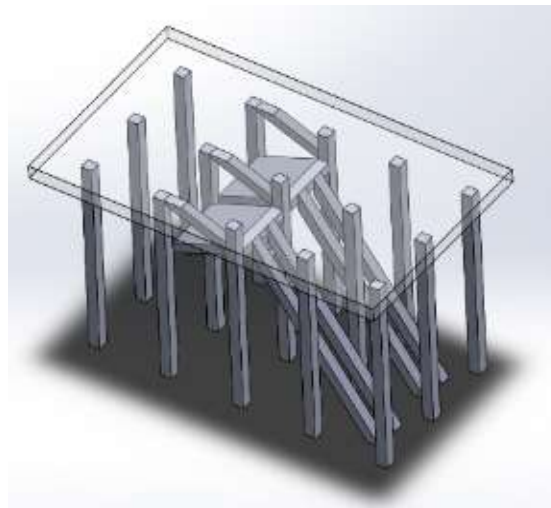


Figure 18: Transparent Municipal Pier Design (MIDN Connor Prince, SolidWorks, 2019)

Currently the deck resides at 12 feet above the MLLW level in order to account for both sea level rise and any possible storms in the area. After finding the bathymetry in MATLAB, it was seen that the height of the sea floor varies with the side to the west of the pier having a steeper slope. This pier may have been ahead of its time in 1929, but that is no longer the case for the decaying 90-year-old structure. It is no longer common practice to place this many piles or to have timber encased squared piles, due to better structural engineering methods (Professor David Kriebel Ph.D., P.E., 2020). Additionally, wave baffle designs have changed and placing the baffle mid structure would no longer be recommended as maneuvering a boat under the pier or using divers to repair the breakwater would be dangerous and difficult (Professor David Kriebel Ph.D., P.E., 2020). Design alternatives will incorporate modern materials, methods, and designs.

Design Alternatives/Analysis

The goal for creating a successful design alternative was to stay in line with the problem statement, based on the requirements set by the NPS. The NPS requested a new pier due to the severe concrete spalling and deterioration of the current pier. With the criteria considered from the problem statement, several categories of design alternatives were formed. Initially, the economic piece was addressed in order to meet the standards of the NPS. In doing this, the width of the pier was examined closely, which is sixty-feet, and this led to the decision to narrow the width of the pier. Narrowing the pier allows for less piles to be used, less deck area, and less overall cost of materials. When considering narrowing the deck and reducing the number of piles, it was decided to base the pier's new width of forty-two feet off the standard three-point turn radius of a San Francisco ambulance. This is to allow for a time friendly emergency response. The length of the standard ambulance is just under twenty seven-feet so the forty-two foot length was completely justified in regards to an ambulance making a successful three-point turn at any point along the pier. To legitimize the use of three piles against the current five, structural calculations have been completed and include: dead, live, and point loads on the deck as well as load calculations on the piles caps and piles. With the structural calculations completed, the minimum thickness of the deck was determined based on its ability to meet the established factor of safety of 1.6 and 1.2 with live and dead loads from Professor David Kriebel Ph.D., P.E. of the United States Naval Academy, respectively. The end goal was to minimize the thickness of the deck in order to minimize cost. Costs will be cut primarily by reducing the width of the pier to forty-two feet. This decision was previously discussed as it pertains to the dimensions of a standard EMS vehicle and its three point turning radius. The reduction of cost is justified based on NPS's request to keep the pier its current length and shape. The revised pier deck can be seen below in Figure 19.

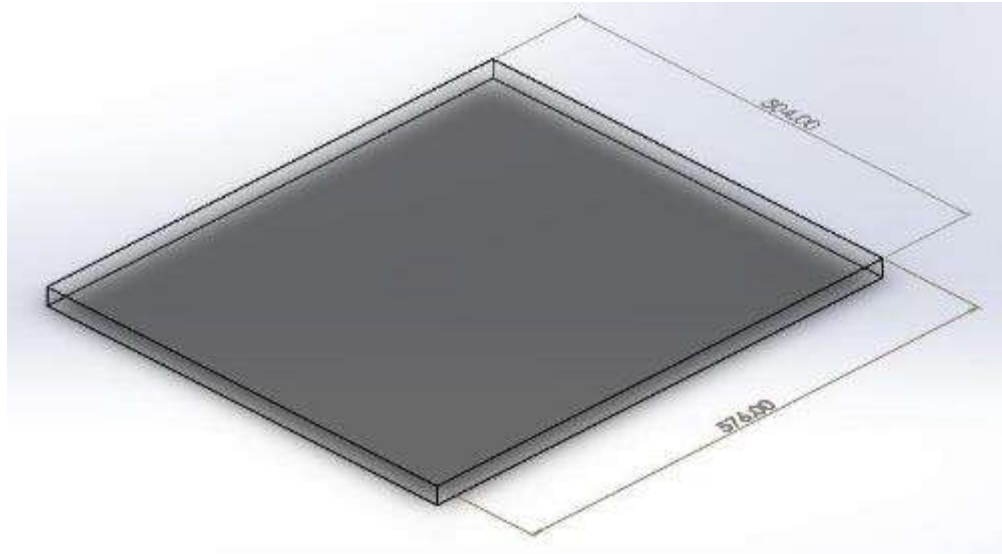


Figure 19: Typical Pier Deck Slab (MIDN Connor Prince, SolidWorks, 2019)

Another major design consideration was the breakwater system that will be used. It was broken into two categories, submerged and unsubmerged. The submerged options are a stone reef and reef balls. The submerged breakwater options can be seen below in Figures 20 and 21.

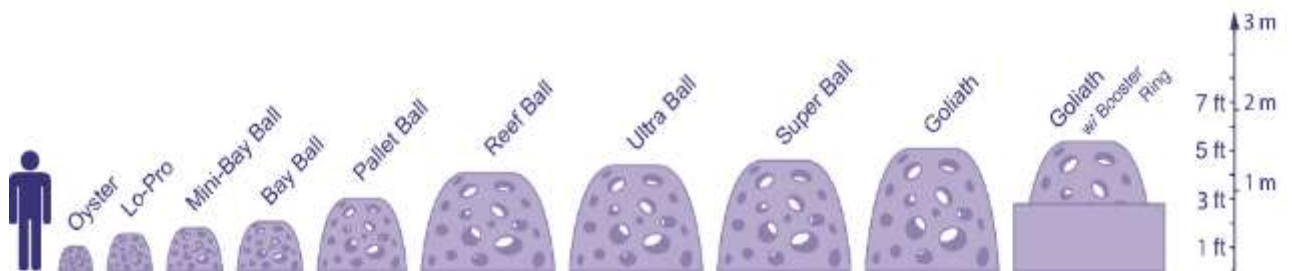


Figure 20: Reef Ball Model Sizes (Reef Ball Foundation, 2016)

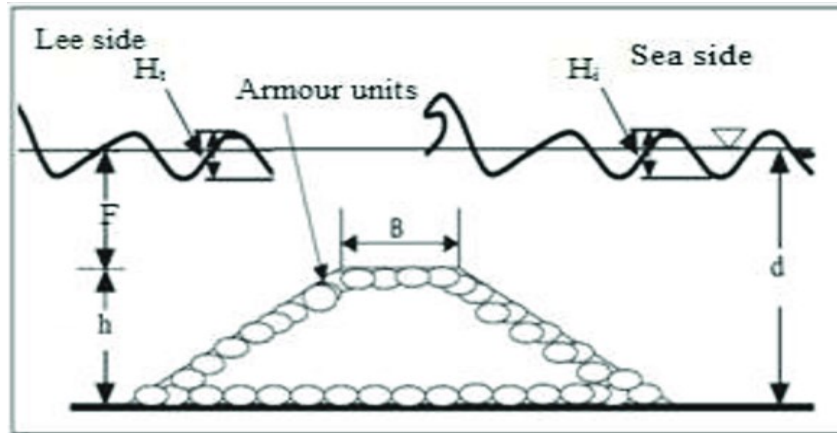


Figure 21: Cross Section of Submerged Reef Breakwater (Sreeranga, Sindhu & Shirlal, Kiran & Manu, 2015)

The use of a submerged breakwater system will not only enhance the environment in the water with attracting all kinds of wildlife, but is predicted to also help in attenuating waves. The unsubmerged breakwater system is broken up into two categories, impermeable and permeable. The unsubmerged breakwater options can be seen below in Figures 22 and 23.

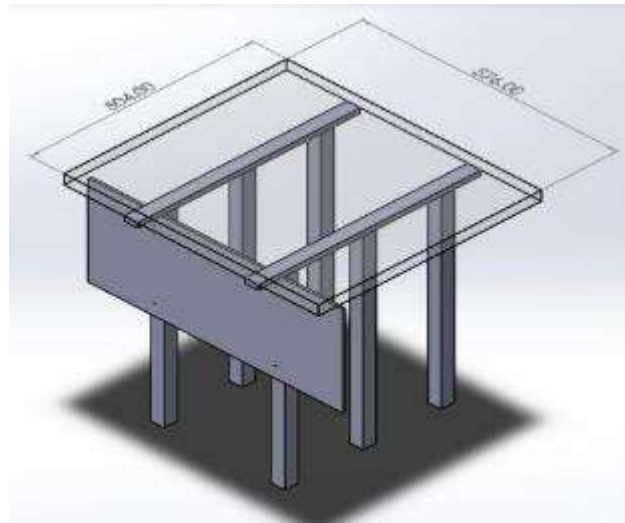


Figure 22: Impermeable Breakwater, spans entire length (MIDN Connor Prince, SolidWorks, 2020)

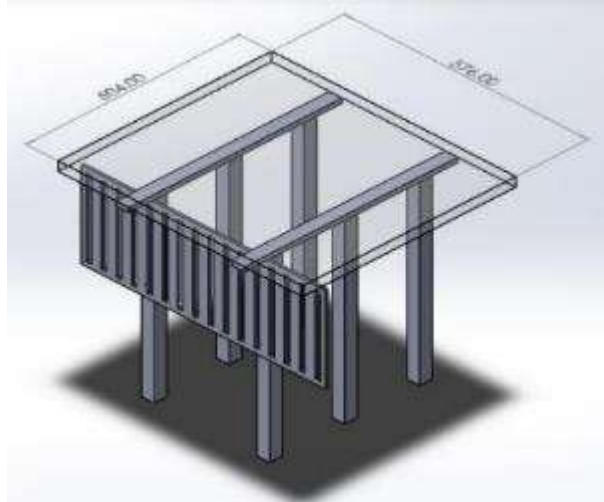


Figure 23: Permeable Breakwater (MIDN Connor Prince, SolidWorks, 2020)

The use of an impermeable breakwater is predicted to attenuate waves better than a permeable one will because less water will pass through the solid concrete surface. When looking at the types of breakwater systems, focus was put on the functionality, cost, reparability, and environmental effectiveness. This can be seen below in Tables 2 and 3.

Table 2: Breakwater Decision Matrix (MIDN Elvin and Prince, 2020)

Decision Matrix - Breakwater					
Weight (%)	30%	30%	20%	20%	
Designs	Cost	Functionality	Reparability	Environmental Effectiveness	Total
Impermeable	5	4	5	0	3.70
Slotted	4.5	3.5	4	0	3.20
Impermeable with Stone Reef	4	4.5	3	3	3.75
Impermeable with Reef Balls	4	4.5	3.5	5	4.25
Slotted with Stone Reef	3	4	3	3	3.30
Slotted with Reef Balls	2.5	4	2	5	3.35

Table 3: Pier Deck Decision Matrix (MIDN Elvin and Prince, 2020)

Decision Matrix - Pier Deck				
Weight (%)	50%	25%	25%	
Designs	Cost	Civilian Capacity	Aesthetics	Total
60 feet (Original)	3	5	4	3.75
42 feet	5	3.5	3	4.13

The decision matrix was utilized to determine which alternative design is the most effective at

fulfilling the designated tasks laid out in the problem statement. A 1-5 scale was used where 1 is the least effective at accomplishing the desired task, while 5 is the most effective at accomplishing the designated task. It is predicted that the use of an impermeable breakwater system along with the use of reef balls will allow for the best overall results. Some driving factors in the breakwater system were the location, size, and material of the system itself. Based on the predicted results of the decision matrix, the use of both a stone reef and a permeable breakwater system has been ruled out. A scaled model of the impermeable breakwater system has been constructed and will be tested in the future to validate the decision-making process.

The final idea moving forward with the design alternatives is the change in deck height. It is predicted by 2100 that there will be a 3.4-foot sea level rise. With that being said, it is clear that raising the height of the deck will mean an increase in the length of the piles resulting in more money used, but if the pier is underwater then that will defeat the purpose of having one in the first place. A design matrix was not created for this scenario due to the clarity in the choice that deck height must increase for a positive outlook in the future.

Lab Tests

In order to determine whether the alternative design was more efficient at attenuating waves than the original, the models underwent wave testing in the coastal wave tank at the United States Naval Academy. As seen in Figure 24, a 2 ft. wide channel was constructed in 15.4 inches of water that stretched almost the full length of the coastal wave tank. The model was positioned in between two capacitance wave height gages. The gage in front of the model analyzed the wave heights as they entered the channel and the gage behind the model examined the attenuated wave heights.



Figure 24: Wave Channel with Pier and Capacitance Wave Height Gages (MIDN Ian Myers, 2020)

Using the peak wave period of 3 seconds, gained from the wind-wave analysis, the data was then extrapolated to include periods of 4.5 and 6.0-second periods in order to test the models under varying wave conditions. There was no need to scale the periods, as they do not change in the scaling process. The longer periods yielded longer wavelengths, which simulated waves created during storms. In order to achieve varying wave heights during a run, the span of the wave generator was randomized, but averaged over the 70-second run time. Altering the span for each individual wave allowed for a slightly different energy level and thus different wave height. To ensure that none of the waves broke before interacting with the breakwater system, the average span was changed to the maximum percentage that would not allow breaking waves. These max

percentages can be seen below in Table 4 along with the fifteen runs conducted on the model. Each wave period had five runs and the same average span; however, in order to test the largest amount of wave conditions, each of the five runs had a different bin of random alterations to the span of the wave generator.

Table 4: Lab Test Groups (MIDN Russell, 2020)

Test Group	Wave Period (sec)	Bin	Span (%)
1	3	A	60
2	3	B	60
3	3	C	60
4	3	D	60
5	3	E	60
6	4.5	A	70
7	4.5	B	70
8	4.5	C	70
9	4.5	D	70
10	4.5	E	70
11	6	A	90
12	6	B	90
13	6	C	90
14	6	D	90
15	6	E	90

Each test ran for 70 seconds and in order to achieve the best-generated wave conditions, the test was followed by a 5-minute break, which allowed the water in the tank to settle. Although each test had 70 seconds worth of data, only the data from the 20-second mark to the 60-second mark was analyzed because prior to 20 seconds, the waves had not propagated across the entire wave tank and after 60 seconds, same wave data was misconstrued due to reverberation from the back

wall. Using the 40 seconds of data, wave heights were examined before and after the model in Excel. In excel, the standard deviation of both sets of wave heights was calculated in order to yield two variables. Following the calculation for standard deviation, the transmission coefficient was calculated using Equation 4.

$$K_t = \frac{\sigma_{after}}{\sigma_{before}} \quad (4)$$

where K_t is the transmission coefficient, σ_{after} is the standard deviation of the wave heights after the breakwater, and σ_{before} is the standard deviation of the wave heights before the breakwater. Following the calculation for the transmission coefficient, the wave period for that test group was used to calculate the deep-water wavelength using Equation 5.

$$L_o = \frac{(g*12)*T^2}{2*\pi} \quad (5)$$

where L_o is the deep-water wavelength in inches, g is the gravitational constant of 32.2 ft/s², and T is the wave period in seconds. The gravitational constant was multiplied by 12 in order to convert it to in/s². After finding the deep-water wavelength, it was used along with the depth of the wave tank to find the wavelength using Equation 6.

$$L = L_o * \sqrt{\tanh\left(\frac{2*\pi*D}{L_o}\right)} \quad (6)$$

where L is the wavelength in inches, L_o is the deep-water wavelength in inches, and D is the depth of the wave tank in inches. The final calculation of the wave data was dividing the depth of the wave tank by the wavelength to get a ratio. All of the calculations were completed for each test group and the values can be seen in Table 5 below.

Table 5: Analyzed Wave Data (MIDN Russell, 2020)

Test Groups	K_t	T (sec)	L_o (in)	D (in)	L (in)	D/L
1	0.290817	3	553.477	15.4	230.252	0.0669
2	0.210019	3	553.477	15.4	230.252	0.0669
3	0.276881	3	553.477	15.4	230.252	0.0669
4	0.363670	3	553.477	15.4	230.252	0.0669
5	0.265443	3	553.477	15.4	230.252	0.0669
6	0.551770	4.5	1245.324	15.4	346.781	0.0444
7	0.564151	4.5	1245.324	15.4	346.781	0.0444
8	0.531845	4.5	1245.324	15.4	346.781	0.0444
9	0.559147	4.5	1245.324	15.4	346.781	0.0444
10	0.599028	4.5	1245.324	15.4	346.781	0.0444
11	0.696689	6	2213.909	15.4	462.692	0.0333
12	0.749080	6	2213.909	15.4	462.692	0.0333
13	0.751156	6	2213.909	15.4	462.692	0.0333
14	0.749080	6	2213.909	15.4	462.692	0.0333
15	0.671773	6	2213.909	15.4	462.692	0.0333

Using the values for K_t and D/L a graph was created, seen in Figure 25, which shows that with the longer wavelengths the transmission coefficient was high while at shorter wavelengths the transmission coefficient was low. This means that under normal wave conditions, the breakwater system is effective at attenuating waves; however, with rough wave conditions and especially with waves generated by storms, the breakwater system has little to no effect on diminishing the waves entering the lagoon.

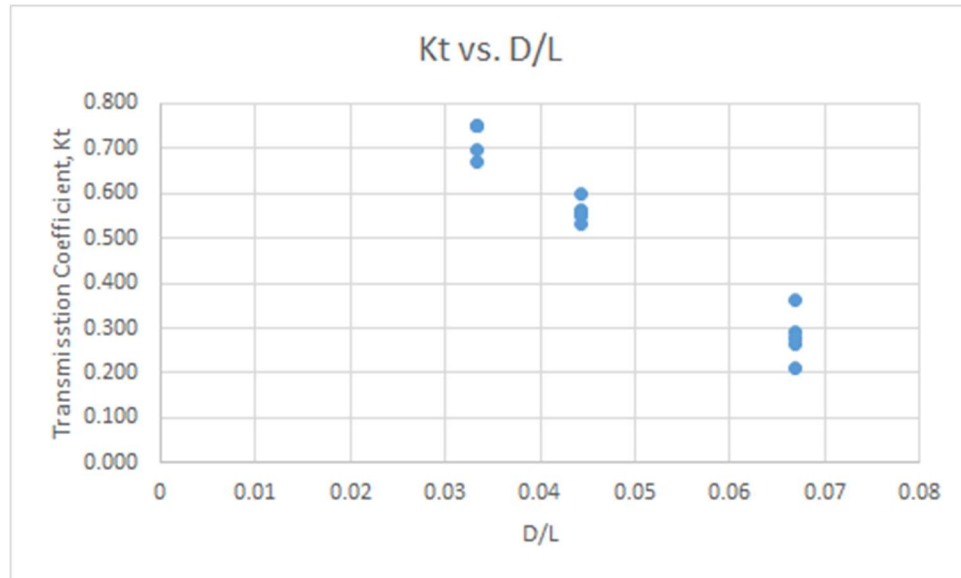


Figure 25: Analyzed Wave Data (MIDN Russell, 2020)

Structural Calculations

Deck and Pile Cap

Detailed structural calculations have been completed to analyze the forces on the deck and pile caps as well as the required amount of reinforcement to sustain forces such as dead, live, and point loads. The purpose of reinforcement is to increase the tensile strength in the deck and pile caps. As stated previously, concrete performs very well in compression and poorly in tension, which is the reasoning behind finding these reinforcement calculations. Figures 26 and 27 below depict the deck, split into six, 24 ft. by 14 ft. slabs, with three pile caps and the pile cap with three piles.

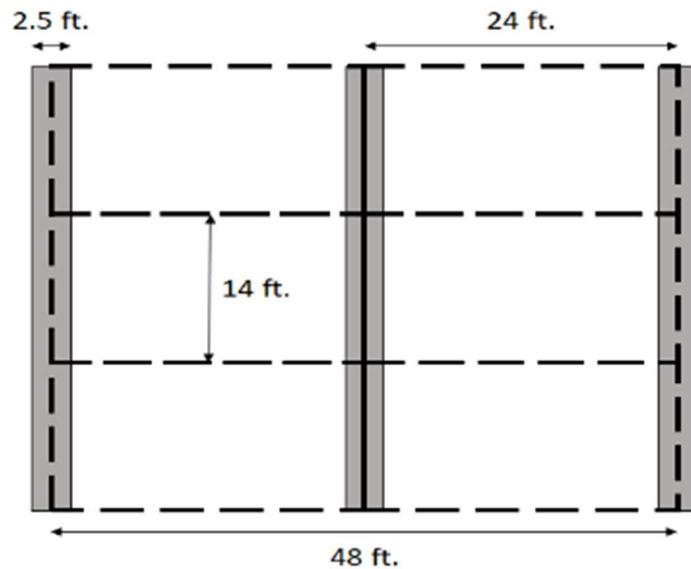


Figure 26: Deck slabs and pile caps (MIDN Russell, 2020)

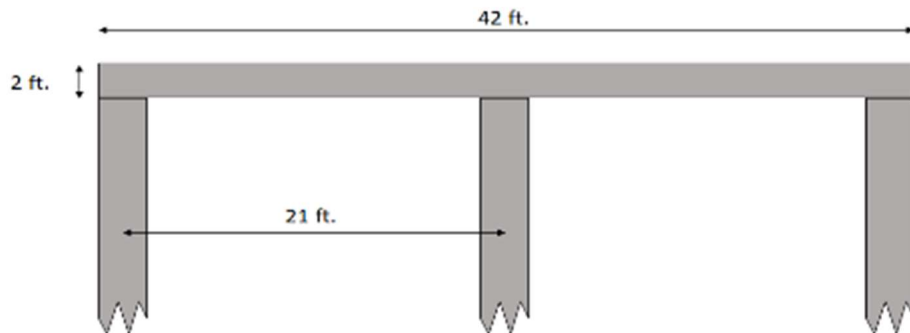


Figure 27: Pile Cap and three piles (MIDN Russell, 2020)

The cross section of the deck analyzed measures 24ft x 0.5ft x 42ft in reference to length, depth, and width. These dimensions were achieved through careful analysis of multiple loads such as point, live, and dead loads and their effects on the deck slab as well as how the slab would be laid and the need for an EMS vehicle to make a three-point turn. While the latter factors were used to determine the width and length of the slab, the height was chosen through load and strength analysis. A point load is defined as “an established load (force), located at a certain point on a supporting structure” (Builder, 2020). For this load, the front and back tires of a standard San Francisco EMS ambulance were used. A dead load is defined as “a load that relatively does not change over time (Vidyakar, 2014). A dead load in this instance is the weight of the deck itself. The last type of load analyzed was the live load. A live load is defined as “a load that can change over time” (Vidyakar, 2014). In this case, the live load is the amount of people standing on the cross section of the slab that has been previously defined. The remaining load calculations are explained in Appendix D. Following the load analysis, a design strength examination was conducted in order to determine the amount of reinforcing steel needed. Using equations and processes from Ambrose and Tripeny, *Simplified Design of Concrete Structures* the analysis was completed and an area of the steel was found. In order to find the amount of steel cables needed, Harris Supply Solutions’ Rebar sizes and stock was referenced, thus yielding an appropriate number of steel cables for the given area. This analysis as well as the load and strength design solutions can be found in Appendix D.

Similar to the deck analysis, the pile caps underwent load and strength design analysis. Despite using the same equations and processes for both examinations, the pile caps differed in two ways. First, the load analysis did not include a point load as the total load that the deck experienced was used as the live load that the pile cap underwent. Secondly, the dimension of the pile cap that was altered was the width instead of the height because it had to be wide enough to cover pile dimensions as well as give enough space for the ends of two deck slabs.

Each analysis underwent three trials in order to find the best height for the deck and width for the pier. Ultimately, the best deck height was found to be 6 in. and a reinforcing steel area to be 1.54 in², thus yielding two #8 steel cables for a 1-ft. section. The pile cap analysis yielded a width of 7 in. and a reinforcing steel area of 5.17 in. Unfortunately, this width was too small compared to

the 2.5 ft. piles and did not provide enough space for the ends of deck slabs to be placed. This problem was solved by increasing the width and thus the reinforcing steel area by a factor of 4.743. By increasing the pile cap width to 35 in. and the area of reinforcing steel to 24.52 in² plenty of space for the ends of deck slabs in addition to the piles was made. With this new area of reinforcing steel the number of cables for the pile cap was 11 #14 bars. The calculations and explanation of results can be found in Appendix D.

Piles

Using Civil engineering concepts from Soils and Foundations, 3rd edition (Liu and Evett, 1991), the length and diameter of piles required for a safe Pier were determined. From the design alternatives section, it was determined that rows of 3 piles would be used for the alternative design. Considering this, each bent of piles must structurally support a concrete deck 42 ft. long, 24 ft. wide, and 0.5 ft. thick. Additionally the three piles must support a bent cap. The dimensions of the pile cap were 42 ft. long, 2.5 ft. wide, and 2 ft. thick. The piles had to be designed to accommodate both of the loads safely. As shown in Appendix E the two loads were combined and a factored load for pile design purposes was calculated.

The NPS drilled for an in depth sediment of the pier in 2020 but this data was not available at the time of this report. Due to the lack of knowledge about seafloor sediments the pile design process assumed a seafloor of submerged sand with a unit weight of 60 lb./ft³ (Liu and Evett, 1992). This was an important early decision because every pile must be driven into the submerged sand seafloor in order to support a load. The piles were designed as purely concrete since they experience mainly compression and steel corrosion in the marine environment was a point of failure for the current standing Municipal Pier. The load of the pile cap, deck, and pile itself will need to be supported by the seafloor while the buoyancy of the submerged portion of the pile will also help resist the load. Total load resistance of the pile is a combination of the bearing capacity of the driven pile tip and bearing capacity from the adhesion between seafloor sediment and the pile wall. The pressure on the pile walls does not linearly increase with depth because the pile has a critical length in the sea floor where the pressure exerted on the walls of the pile becomes constant with increasing depth.

Setting the design load equal to the sum of the friction and tip resistance yielded the length the piles must be driven for a variety of pile shapes. The equations and processes supporting these findings are located in Appendix E. Below Tables 6 and 7 summarize the results.

Table 6: Circular pile diameter and length requirements (MIDN Myers, 2020)

Circular Piles				
Diameter (ft)	1.5	2	2.5	3
Driven Length (ft)	253	136	79	45
Total Pile Length (ft)	298	181	124	90
Total Volume (ft³)	527	569	609	636

Table 7: Square pile base and length requirements (MIDN Myers, 2020)

Square Piles				
Base Length (ft)	1.5	2	2.5	3
Driven Length (ft)	199	105	59	31
Total Pile Length (ft)	244	150	104	76
Total Volume (ft³)	549	600	650	684

For construction of the alternative design, piles would be precast and mass produced to be the same size and shape. While not every pile would be exposed to an equal share of the design load, in Appendix E the piles were designed under the constraints of the pile which takes the majority of the load in order to avoid overloading failure. From the results, since the square piles were found to require more total volume they were ruled out for the alternative design in favor of the circular piles. Additionally, the current design has concrete jacketed timber piles that are square. The square piles have eroded badly over the years as shown in the site visit photos in appendix C. Lastly, according to Liu and Evett in their textbook *Soils and Foundations* (1992), large diameter precast piles are typically of cylinder shape. While the 3 ft. circular pile requires shorter total length, the extra material due to the total volume would increase the manufacturing price. Minimizing cost if possible has been emphasized as a design goal and the 2.5 ft. diameter pile has a more economical total volume. Furthermore, the total length of the 2.5 ft. diameter circular pile at 124 ft. long is more feasible for transportation and installation than the smaller diameter piles of greater lengths. For these reasons the 2.5 ft. diameter, 124 ft. long circular piles were used for the alternative design.

Wave Transmission and Lateral Forces

Wave transmissions are crucial to the breakwater design so that it attenuates oncoming waves effectively. Determining the lateral forces of these waves gives the required parameters in order for the pier to withstand the environmental conditions. Figure 28 shows lowest and highest sea levels of the years 2020 and 2100. On the left, are the levels in the present day and on the right are the predicted tide levels in the year 2100.

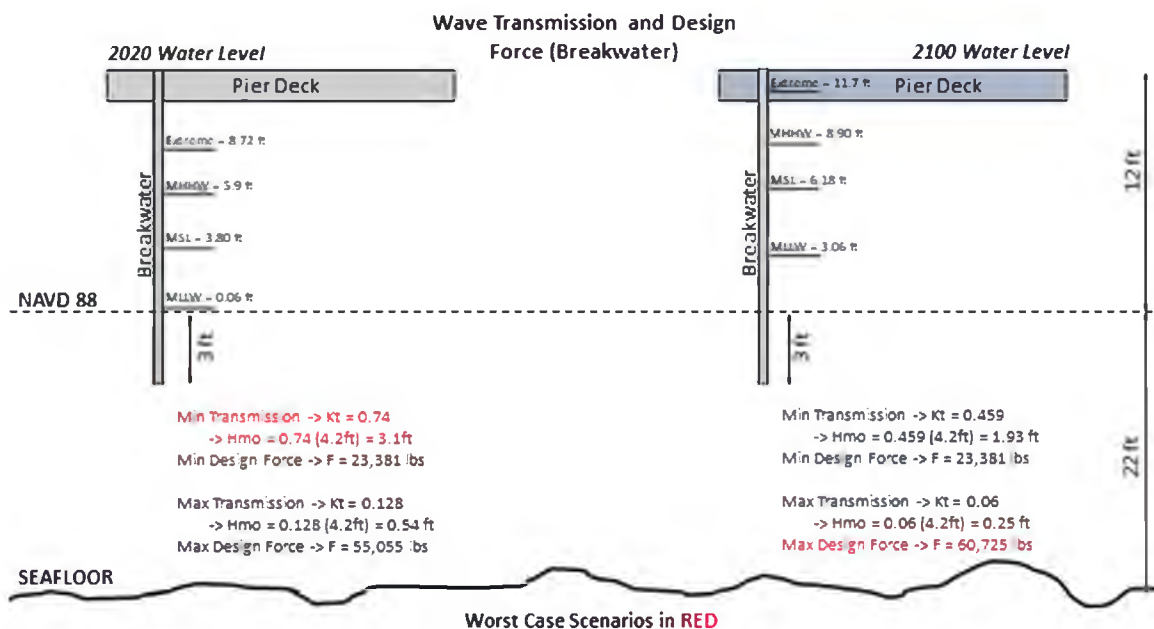


Figure 28: Depiction of water levels for the year 2020 (right) and the year 2100 (left) (MIDN Ostrowski, 2020)

Analyzing the effectiveness of this breakwater design involved calculating the wave transmission and lateral forces on the barrier. Using ocean wave mechanic concepts found in Appendix D, the breakwater performance was calculated theoretically. Below are the plots of wave transmission and lateral forces on the barrier.

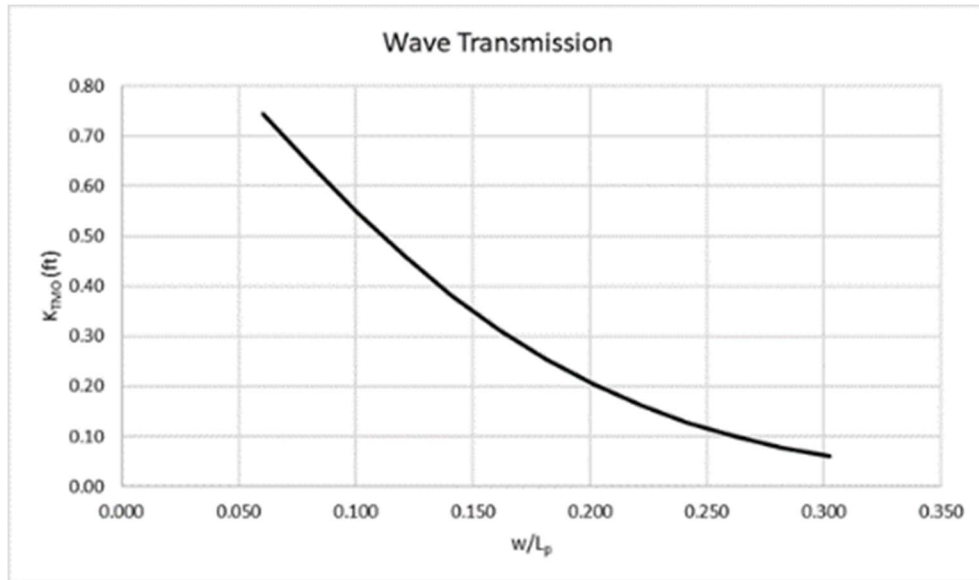


Figure 29: Wave transmission of breakwater design as draft increases (MIDN Prince, 2020)

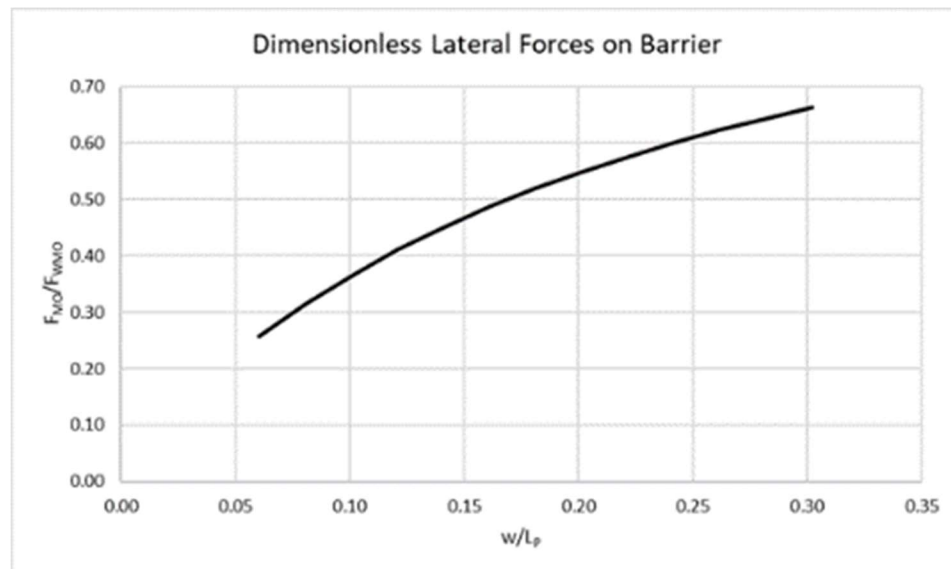


Figure 30: Lateral forces on breakwater as draft increases (MIDN Prince, 2020)

Figure 29 represents the wave transmission as they pass through the pier and Figure 30 represents the lateral forces on the breakwater. During the mean lower low tide of 2020, the breakwater would have a draft three feet and the breakwater would produce the worst wave transmission of 7.4. As the draft increases with tides and future sea-level rise, the wave transmission improves drastically. The best results, 0.06, occurred at a draft of 15 feet, which

represents the predicted extreme high tides of 2100. When viewing the lateral forces on the breakwater, the results concur with those of the wave transmissions. The minimum forces would occur during the mean lower low tide of 2020, which was 23,381 lbs. These would steadily increase as the water level rises to the extreme high tides predicted of 2100, which resulted in a maximum force of 60,725 lbs. This, however, will not be much larger than the forces that would occur during the present day at extreme water levels of 55,055 lbs. To conclude, the alternative design would theoretically be able to perform better than the existing pier, especially if the sea level rises as predicted.

Cost Analysis

NPS carried out a replacement pier construction cost estimation in 2018, which was produced by Anchor QEA, LLC on behalf of Davido Consulting Group Incorporation (2018). The construction costs for eight potential alternatives were estimated. One alternative in particular was very close in size and shape to the alternative design that has been researched in this report. The cost estimate listed it as alternative 7 of 8. The cost estimate describes the alternative as the following:

Alternative 7 would consist of complete replacement of the existing Muni Pier with a narrower version of the existing pier. This alternative would include fewer piles per bent and would possibly use larger diameter piles to eliminate the need for batter piles. This would result in a 25- or 30-foot-wide deck, which would be one-third to one-half, respectively, of the historic width. This alternative would have reduced visual quality from the historic pier. Multipurpose use is possible under this alternative. A perforated wall attenuator or hanging baffle would be used. Replacement of only 10 fender piles is included, because this design is narrower than the current pier structure. Removal of debris on the Bay bottom, which was found during the 2017 bathymetric survey, is not necessary. The pier would have a seismic design under this alternative (Davido Consulting Group, 2018).

The only difference between alternative 7 and the alternative researched in this report is the difference in deck width; 25-30 ft. in alternative 7 as opposed to 42 ft. in the selected alternative design. The cost estimation was in depth and included many variables. Using the Work Breakdown Structure (WBS) method, the items listed in Table 8 were prepared by Anchor QEA, LLC (2018). Mobilization included all field staging construction costs, mobilization, demobilization, and environmental management best practices. The Piles item referred to the cost for installing new piles. Deck and soffit framing estimated the cost of replacing pile bent caps and beams. Deck Repair was the price for replacing the deck surface, benches, and guardrails. Lighting included the process of replacing the electrical system and light standards. Potable water referred to the cost for replacing the potable water system. Wave attenuation defined the effort of replacing the wave baffle system. Finally, pier demolition estimated the price of completely demolishing the existing Municipal Pier.

Table 8: Summarized WBS asset element breakdown of 2017 updated estimates of construction costs for alternative 7 (Davido Consulting Group Inc., 2018)

Item	7
Mobilization	\$390,000
Piles	\$6,151,000
Deck and Soffit Framing	\$5,148,000
Deck Repair	\$13,755,300
Lighting	\$286,000
Potable Water	\$0
Wave Attenuation	\$10,920,000
Pier Demolition	\$6,210,813
Sub-Total Estimated Direct Construction Costs	\$42,861,113
Mark-Ups and Add-Ons	\$38,612,681
Total Estimated Net Costs of Construction	\$81,473,793

In 2017, the total estimated cost of replacing the existing pier with alternative 7 was \$81,473,793. Since the alternative researched throughout this report was 42 ft. wide it was decided that an extra \$5,000,000 was needed to conservatively compensate for the extra amount of deck and soffit framing material needed. The 2017 cost estimate for the 42 ft. wide deck was calculated as \$86,473,793. This estimate was adjusted for inflation for a 2020 cost estimate.

Inflation Calculator

If in (enter year)

I purchased an item for \$

then in (enter year)

that same item would cost:

Cumulative rate of inflation:

\$91,058,188.15

5.3%

CALCULATE

Figure 31: 2020 alternative design cost estimate (US Inflation Calculator, 2020)

The comprehensive cost of tearing down the existing Municipal Pier and installing the proposed alternative pier in 2020 was found to be \$91,058,188.15. As seen in Figure 31 above.

Moving Forward

Following the structural calculations, testing for the alternative design was planned as well as an updated design matrix and cost analysis. Due to the corona virus, the United States Naval Academy's spring semester was cut short and the coastal wave tank was closed. Without the ability to test the new design, the decision matrix and cost analysis were based on research, theoretical analysis, and the structural calculations. As previously mentioned, the pier has a unique curvature, which is due to the use of multiple radii that span the length of the pier. The next step moving forward would be to analyze the radii and use them for the construction of the new pier. The radii would determine how the piles are drilled and the deck slabs are laid. The shape of each slab would affect the implementation of the deck slabs. As seen in Figure 26 the original 24 ft. by 42 ft. slab was split into three, 24 ft. by 14 ft. slabs on the assumption that this would aid in the implementation process especially if they were pre-cast. By pre-casting the slabs each one could be specifically designed for its section and with a smaller size slab the curvature could be more accurately represented. The last step moving forward would be the removal of the current pier. The cost and environmental problems associated with the removal would be the biggest factors to consider.

Conclusion

The NPS has requested a design alternative to the current San Francisco Aquatic Park Pier. In doing this, emphasis was put on staying in line with guidelines set by the NPS in the problem statement as well as the codes for the city of San Francisco. Design alternatives have been fabricated and tested that fulfill the needs set by the NPS. A site visit was conducted during mid February to meet with the NPS and see the conditions of the current structure. During this visit, it was clear that the primary issue with moving forward with the reconstruction process would be the lack of funding from the city of San Francisco. A presentation was made and performed with the present NPS staff. This helped to hone in on the severity of issues with the pier and the importance of coming up with a successful alternative design. After returning from San Francisco, an in depth look at the test results from the coastal tank were analyzed. From this, it was observed that the current pier model was effective at attenuating normal wave conditions. However, when conditions that simulate a storm like condition were created, the current pier design was not as effective at attenuating waves as it could be. Based on these results, another model was created in order to fulfill all of the needs of the problem statement, but also outperform the old pier design in every way. Unfortunately, due to COVID-19, testing was not able to be conducted on the alternative design model of the pier. In depth structural calculations of forces acting on the pier deck, pile caps, and piles were completed in order to try and move further in the design process while not being able to physically test the alternative design. Using NPS documentation, the comprehensive cost of tearing down the existing Municipal Pier and installing the proposed alternative pier in 2020 was found to be \$91,058,188. The next step moving forward is to run tests on the alternative design and make adjustments based on the results in order to make the most effective alternative design possible.

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Acknowledgements

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Appendix A

The following equations and explanations continue the wind-wave analysis process after Equation 1 in the report and end right before the significant wave height and peak wave period need to be found. The following equations and explanations were taken from the Naval Academy Coastal Engineering textbook, *Coastal Engineering* by Professor David Kriebel.

Using the value found in Equation 1, the time required for waves crossing a fetch under a wind velocity to become fetch limited was calculated with Equation A1.

$$t_{x,u} = 77.23 \frac{X^{0.67}}{u^{0.34} g^{0.33}} \quad (\text{A1})$$

Where X is the fetch in meters, u is the wind velocity in m/s, g is the gravitational constant 9.81 m/s^2 , and $t_{x,u}$ is the time in relation to fetch and wind speed in seconds. The value determined was 6733.5 seconds. Using the new time value, Figure 13 again as well as Equation A2 was used to determine a new wind speed ratio.

$$\frac{U_t}{U_{3600}} = 1.5334 - 0.15 \log_{10} t \quad (\text{A2})$$

Where U_t is the wind speed for the time variable found from Equation 3 in m/s, U_{3600} is the one hour wind speed in m/s, and t is the time variable determined by using Equation A1 in seconds. After the ratio was found, the wind speed for the time at 6733.5 seconds was calculated by multiplying by the 1-hr. wind speed. Using this value, the drag coefficient was calculated using Equation A3.

$$C_D = 0.001(1.1 + 0.035 U_{6733.5}) \quad (\text{A3})$$

Where $U_{6733.5}$ is the wind speed at 6733.5 seconds in m/s and C_D is the drag coefficient. By obtaining a value for the drag coefficient, the friction velocity was calculated with Equation A4.

$$u_* = \sqrt{C_D * U_{6733.5}^2} \quad (\text{A4})$$

Where u_* is the friction velocity in m/s, C_D is the drag coefficient, and $U_{6733.5}$ is the wind speed at 6733.5 seconds in m/s.

Appendix B

This appendix contains tables from the official State of California Sea-Level Rise Guidance report (State of California Sea-Level Rise Guidance , 2018). Table B1 breaks down probabilities for a variety of sea-level rise possibilities. It also assigns sea level rise values for low, medium, and high-risk aversion scenarios. The criteria for these scenarios are located in Table B2.

Table B1: California Sea-Level Rise Projections (State of California Sea-Level Rise Guidance , 2018)

		Probabilistic Projections (in feet) (based on Kopp et al. 2014)				H++ scenario (Sweet et al. 2017) *Single scenario
		MEDIAN	LIKELY RANGE	1-IN-20 CHANCE	1-IN-100 CHANCE	
		50% probability sea-level rise meets or exceeds...	66% probability sea-level rise is between...	5% probability sea-level rise meets or exceeds...	0.5% probability sea-level rise meets or exceeds...	
			Low Risk Aversion		Medium - High Risk Aversion	Extreme Risk Aversion
High emissions	2030	0.4	0.3 - 0.5	0.6	0.8	1.0
	2040	0.6	0.5 - 0.8	1.0	1.3	1.8
	2050	0.9	0.6 - 1.1	1.4	1.9	2.7
Low emissions	2060	1.0	0.6 - 1.3	1.6	2.4	
High emissions	2060	1.1	0.8 - 1.5	1.8	2.6	3.9
Low emissions	2070	1.1	0.8 - 1.5	1.9	3.1	
High emissions	2070	1.4	1.0 - 1.9	2.4	3.5	5.2
Low emissions	2080	1.3	0.9 - 1.8	2.3	3.9	
High emissions	2080	1.7	1.2 - 2.4	3.0	4.5	6.6
Low emissions	2090	1.4	1.0 - 2.1	2.8	4.7	
High emissions	2090	2.1	1.4 - 2.9	3.6	5.6	8.3
Low emissions	2100	1.6	1.0 - 2.4	3.2	5.7	
High emissions	2100	2.5	1.6 - 3.4	4.4	6.9	10.2
Low emissions	2110*	1.7	1.2 - 2.5	3.4	6.3	
High emissions	2110*	2.6	1.9 - 3.5	4.5	7.3	11.9
Low emissions	2120	1.9	1.2 - 2.8	3.9	7.4	
High emissions	2120	3	2.2 - 4.1	5.2	8.6	14.2
Low emissions	2130	2.1	1.3 - 3.1	4.4	8.5	
High emissions	2130	3.3	2.4 - 4.6	6.0	10.0	16.6
Low emissions	2140	2.2	1.3 - 3.4	4.9	9.7	
High emissions	2140	3.7	2.6 - 5.2	6.8	11.4	19.1
Low emissions	2150	2.4	1.3 - 3.8	5.5	11.0	
High emissions	2150	4.1	2.8 - 5.8	5.7	13.0	21.9

Table B2: Risk Decision Framework (State of California Sea-Level Rise Guidance , 2018)

RISK CONSIDERATIONS & EVALUATION	Consequences of Impact or Disruption	LOW Minimum Disruption, Limited Scale and Scope	MEDIUM TO HIGH Inconvenience, but Limited in Scope and Scale	EXTREME Unacceptable Risk and/or Extensive Scale and Scope
	Adaptive Capacity	• Future flexibility maintained • People or systems readily able to respond or adapt	• Limited future flexibility	• Irreversible • Threat to public health and safety
	Who or What is Affected?	• Low impact on communities, infrastructure, or natural systems	• Communities, systems, or infrastructure readily able to adapt or respond to change	• Vulnerable populations • Critical infrastructure • Areas of economic, historic, or cultural significance
	Economic Impacts	LOW	MEDIUM	HIGH
EMISSIONS SCENARIO EVALUATION	Pre-2050	RCP 8.5 (high emissions)	RCP 8.5 (high emissions)	RCP 8.5 (high emissions)
	Post-2050	← EVALUATE RCP 2.6 AND RCP 8.5 (low emissions and high emissions) →		
SLR PROJECTIONS SELECTION		LOW RISK AVERSION	MEDIUM-HIGH RISK AVERSION	EXTREME RISK AVERSION

Appendix C: Site Visit

The Pier Builders were able to visit Municipal Pier during a site visit from the 18th through 20 February 2020. This trip could not have been possible if it was not for a generous donation from Mr. Volgeneau. Located in this appendix are photos that give the reader a frame of reference for the aquatic park and additionally they show the status of the Pier today. During the visit, the Pier Builders were able to meet with the Project Managers and main caretakers of Municipal Pier. On 18 February, the students arrived around midday and proceeded to get their bearings in San Francisco and in the area around Aquatic Park. The following day, Municipal Pier Project Manager Michael Bell set up a meeting between the NPS and the Pier Builders. This meeting gave the Pier Builders a valuable opportunity to present findings that had been processed up to that point in February, and to receive crucial feedback from those who know Municipal Pier best. Attending the presentation from the NPS were: Michael Bell, Phil Erwin, and Jeff Morris. As stated previously Mr. Bell is the Project Manager, while Phil Erwin is the Chief of the facility and ship management division, and Jeff is the supervisory facility and historic ship operations specialist. The NPS delegation gave the Pier Builders some previously unknown feedback. The feedback given included some historical significance to the pier in that it protects the cove from erosion along with the historic ships that are docked right next to the pier on the landward side.

The last evolution on Wednesday that the Pier Builders took part in was a boat tour of the pier. Walker, Zach, Luke, and Connor went out in a boat with one of the NPS employees and got an up-close look at the deterioration of the pier. Here Connor and Luke were able to take pictures of where the pier was failing. Ian stayed back with Michael and walked along the deck of the pier, while taking pictures of some of the deterioration from up top.

On Thursday, the Pier Builders met with Irinia Irving (Ocean and Coastal Resources Program Manager), Diane (Worker at San Francisco Maritime National Historical Park Research Center), and Gina Bardi (Reference Librarian San Francisco Maritime National Historical Park Research Center) at the San Francisco Maritime National Historical Park Research Center. The team arrived in the morning on Thursday at the archival center and started by receiving a welcome brief from the three women. At the conclusion of the brief, Ms. Diane took the Pier Builders on a tour through the basement of the Archives. She showed the Pier Builders some of San

Francisco's history, which included old statues and other figures that were a part of some of the historic ships in San Francisco. Along with old ship parts, Ms. Diane showed us many paintings that depicted the historical skylines of San Francisco, historic ships, and bridges in the city. After the tour, the Pier Builders were taken upstairs to the library section of the Research Center. Here, Ms. Gina showed the Pier Builders different scrolls and diary's that were kept by some of the first people to settle in San Francisco back in the early 1800s. At the conclusion of this tour, the Pier Builders went back to the common area of the Research Center. Here they presented their plans and findings of Muni Pier to Ms. Irina who was primarily interested in the steep drop-off of the seafloor that occurs on the seaside of the pier. Aside from this, the Pier Builders restated what they had presented in their presentation the previous day to the NPS, since Irena was unable to attend the original brief. This was the last evolution that the Pier Builders attended at the Research Center.

Following the trip to the Research Center, the Pier Builders returned to Muni Pier for one last visit. This time the goal was to get wave readings via the sonobuoys. The team was successfully able to get the buoys into the water, but unfortunately, none of the team members were able to connect their phones to the buoys. Therefore, there was no extra data found here on the wave heights or attenuation of the waves.



Figure C1: Broken piles from shear failure (MIDN Ostrowski and Prince, 2020)

The photo above in Figure C1 was taken during the team's boat ride around Muni Pier. The piles here are located at the end of the pier where it opens up into more of a sphere shape above on the deck. At this location on the pier, shear failure was a common theme.

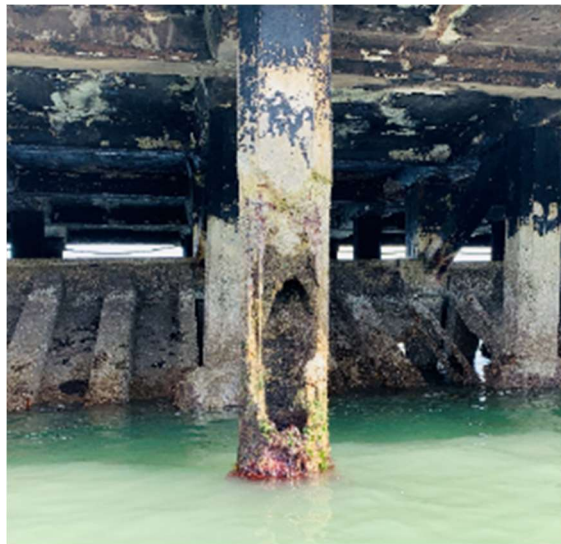


Figure C2: Eroded pile with inner timber exposed (MIDN Ostrowski and Prince, 2020)

Figure C2 shows how some of the piles on the pier are eroding. This picture was taken from the landward side of the pier. The reason that these piles are two different colors is due to an attempt to preserve the piles from erosion. This was a decent short-term solution, but it clearly did not work out in the end.



Figure C3: Spalling wave baffle and eroded piling (MIDN Ostrowski and Prince, 2020)

Figure C3 is another photo from the landward side of the pier. Like the previous image, this also shows the erosion of the piles as well as some spalling on the breakwater structure. At a closer glance where the pile has eroded, one can see the rebar poking out of the cement.



Figure C4: Broken wave baffle (MIDN Ostrowski and Prince, 2020)

The figure above shows the most deteriorated part of the wave baffle. There is all sorts of algae and barnacles that are growing here on the wave baffles, this adds to the erosion problem. An issue from the environmental side of things is once the erosion becomes bad enough, then parts of the pier fall to the seafloor and can harm the marine life in the area.



Figure C5: Spalled Pile with rebar and inner timber exposed (MIDN Ostrowski and Prince, 2020)



Figure C6: Spalled hand railing with steel rebar exposed (MIDN Ostrowski and Prince, 2020)

In Figure C6 the railing of the pier is falling apart due to concrete spalling. When walking down the deck, this part of the pier is closed off to tourists. While there are other parts of the railing that are in bad condition, this is the only point on the pier where the spalling actually created holes in the railing. Figure C7 shows the spalling of the railing (the same part as Figure C6), but just with a view from the deck of the pier. Then, Figure C8 provides a view of the fence that is used to keep pedestrians away from the dangerous parts of the pier deck.



Figure C7: Spalling concrete handrail from the deck of the pier (MIDN Myers, 2020)



Figure C8: Half of the pier blocked from pedestrian use due to failed concrete hand railing (MIDN Myers, 2020)



Figure C9: One of several drilled holes on the pier where sediment drilling occurred (MIDN Myers, 2020)

When walking along the pier there were several areas where holes were drilled into the deck in order to gain sediment samples of the pier. Figure C9 shows one of these areas. Testing was done on these areas in order to determine soil conditions at various points along the length of the Pier.



Figure C10: Sagging decking where repairs were made from a boat collision in the 1940s ((MIDN Myers, 2020)) In the 1940s, the U.S. Navy crashed into Muni Pier, taking out some of the piles along with part of the deck. Figure C10 shows the appropriate repairs that were made in order to fix the pier. In the image, the repairs were made at the point in which a vertical cap is visible. The repairs, however, were not done very well and since they have been completed, the pier has continued to sag a little lower than the rest of the pier. This is an area of concern.



Figure C11: Restricted Area of the pier due to failed hand railing (MIDN Myers, 2020)

Along the seaside of the pier, a large portion is closed off to pedestrians due to the guardrail failure. The NPS members quoted that there are several sections of the railing that could fall off with little force making it unsafe for civilians to stroll along. In Figure 11, a few of said sections can be seen with significant spalling creating holes in the railing.

Appendix D

The following equations and figures show the structural calculations used in the group's analysis of the alternative pier. The documents referenced to aid in the analysis were *Simplified Engineering for Architects and Builders* by Harry Parker and James Ambrose, M.S., *Simplified Design of Concrete Structures* by James Ambrose and Patrick Tripeny, and Harris Supply Solutions' Rebar sizes and stock and www.harrissupplysolutions.com.

In order to properly perform the structural calculations for the deck and pile cap, certain consistent parameters such as the density of reinforced concrete, design live load, modulus of elasticity, and gross moment of inertia were found or calculated in the beginning. The density of concrete, γ_c , the design live load, and modulus of elasticity are known values found in Table D1 below. These values were achieved after detailed and careful discussion with Professor David Kriebel Ph.D., P.E.

Table D1: Deck Analysis (MIDN Russell, 2020)

	Trial 1	Trial 2	Trial 3
Deck Analysis			
γ_c (lb/ft ³)	150	150	150
Design Live Load (lb/ft ²)	100	100	100
Modulus of Elasticity, E (lb/in ²)	6.00E+06	6.00E+06	6.00E+06
Gross Moment of Inertia, I_G (ft ⁴)	0.083333	0.035156	0.010417
Deck Calculations			
Width, b (ft)	42	42	42
Thickness, h (ft)	1	0.75	0.5
Length, L_e (ft)	24	24	24
Width of sliver, b_s (ft)	1	1	1
A_{deck} (ft ²)	1008	1008	1008
V_{deck} (ft ³)	1008	756	504
Deck Weight, DL (lbs)	151200	113400	75600
Deck Live Load, LL (lbs)	100800	100800	100800
Design Deck Load, Q_d (lbs)	342720	297360	252000
Analysis			
Distributed Load			
Distributed Load on 1-ft sliver of Deck, w (lb/ft)	340	295	250
Spacing btw pile caps, L (ft)	24	24	24
Max Bending Moment, $M_{u,DL}$ (ft-lbs)	24480	21240	18000
Max Shear, $V_{DL} = R_1 = R_2$ (lb)	4080	3540	3000
Max Deflection, Δ_{DL} (ft)	2.04E-02	4.20E-02	1.20E-01
Point Load			
Point Load from EMS vehicle for 1-ft sliver (lb)	167	167	167
Distance between point load and support, a (ft)	4	4	4
Max Bending Moment, $M_{u,PL}$ (ft-lbs)	668	668	668
Max Shear, $V_{PL} = R_1 = R_2$ (lb)	167	167	167
Max Deflection, Δ_{PL} (ft)	6.43E-04	1.52E-03	5.15E-03
Combined Loads			
Max Bending Moment, $M_{u,max}$ (ft-lbs)	25148	21908	18668
Max Shear, V_{max} (lb)	4247	3707	3167
Max Deflection, Δ_{max} (ft)	2.10E-02	4.35E-02	1.25E-01
Strength Design			
Thickness= height, h (in)	12	9	6
Width, b (in)	12	12	12
Concrete compressive strength, f_c' (lb/in ²)	4000	4000	4000
Allowable compressive strength, f_{c-A}' (lb/in ²)	3400	3400	3400
Yield strength, f_y (lb/in ²)	60000	60000	60000
Capacity reduction factor, ϕ	0.9	0.9	0.9
Nominal Moment of Inertia, M_n (ft-lbs)	27942	24342	20742
Factors from Table:			
R-factor (psi)	1417	1417	1417
a/d-factor	0.592	0.592	0.592
Percentage of Reinforcing Steel, p	0.0335	0.0335	0.0335
Use R-factore to solve for d			
Depth to reinforcing steel, d (in)	4.44	4.14	3.83
Desired d in order to achieve $d = h - 2$ in.	10	7	4
Use p-factor to find A_s			
Area of reinforcing steel, A_s (in ²)	1.79	1.67	1.54

The gross moment of inertia for both the deck and the pile cap were found using Equation D1.

$$I_G = \frac{b \times h^3}{12} \quad (\text{D1})$$

where I_G is the gross moment of inertia in ft.^2 , b is the width in ft. , and h is the height in ft. For the deck analysis, the width of the deck b was substituted with b_s , which is the width of a sliver of the deck. The nominal value for this width stays constant at 1-ft. because it made the load and strength analysis easier due to converting the square deck into a rectangular simple beam.

Following the establishment of the consistent parameters, the dimensions of the deck were defined. As previously mentioned, the width and length of the deck slab were chosen due to factors such as the need for an EMS vehicle to complete a three point turn as well as ease of implementation. On the other hand, the thickness or height of the deck was altered from 1 ft. in the first trial down to 0.5 ft. in the third. In order to simplify the deck analysis to a simple beam the deck width was split into 42, 1-ft. slivers. One of these 1-ft. slivers was used for the rest of the deck analysis. The area of the deck was calculated using Equation D2.

$$A_{DECK} = Le \times b \quad (\text{D2})$$

where A_{Deck} is the area of the entire deck in ft.^2 , Le is the length of the deck in ft. , and b is the width of the deck in ft. The volume of the deck was then found using Equation D3.

$$V_{DECK} = A_{DECK} \times h \quad (\text{D3})$$

where V_{Deck} is the volume of the deck in ft.^3 , A_{Deck} is the area of the deck in ft.^2 , and h is the thickness or height of the deck in ft. After finding the volume, the density of reinforced concrete was calculated using Equation D4.

$$DL = \gamma_c \times V_{DECK} \quad (\text{D4})$$

where DL is the deck (dead) weight in lbs. , γ_c is the density of concrete in lb./ft.^3 , and V_{Deck} is the volume of the deck in ft.^3 . Following the calculation of the deck weight, the deck live load was found using Equation D5.

$$LL = \text{Design Live Load} \times A_{DECK} \quad (\text{D5})$$

where LL is the deck live load in lbs. , the design live load is in lb./ft.^2 , and A_{Deck} is the area of the deck in ft.^2 . The last calculation for the initial deck dimension analysis was the design deck load and was found using Equation D6.

$$Q_d = (1.2 \times DL) + (1.6 \times LL) \quad (\text{D6})$$

where Q_d is the design deck load in lbs., 1.2 is the safety factor for dead loads, DL is the deck weight in lbs., 1.6 is the safety factor for live loads, and LL is the deck live load in lbs.

The next step in the structural calculations was the distributed load, point load, and combined load analysis for the deck. Figure D1 depicts the distributed load placed upon the deck.

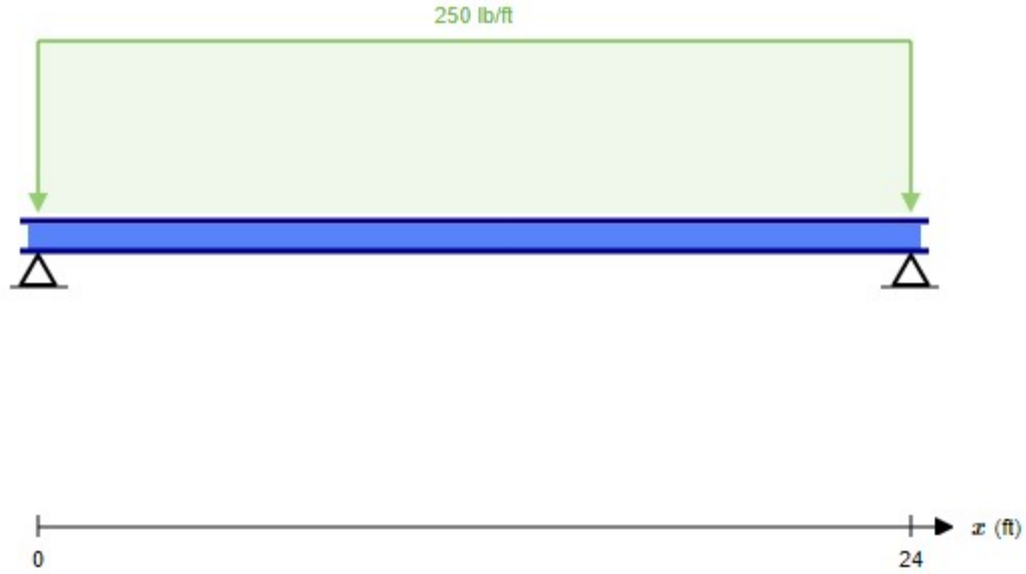


Figure D1: Distributed Load on Deck (MIDN Russell, 2020)

The distributed load on a 1-ft. sliver of the deck was found using Equation D7.

$$w = \frac{Q_d}{Le \times b} \quad (\text{D7})$$

where w is the distributed load on 1-ft. sliver of deck in lb./ft., Q_d is the design deck load in lbs., Le is the length in ft., and b is the width in ft. After finding w , the spacing between pile caps, L , was established at 24 ft. The max bending moment of the distributed load was calculated using Equation D8.

$$M_{u,DL} = \frac{w \times L^2}{8} \quad (\text{D8})$$

where $M_{u,DL}$ is the max bending moment for the distributed load in ft.-lbs., w is the distributed load on a 1-ft. sliver of deck in lb./ft., and L is the spacing between pile caps in ft. Following the moment calculation, the max shear was found using Equation D9.

$$V_{DL} = \frac{w \times L}{2} \quad (\text{D9})$$

where V_{DL} is the max shear for the distributed load in lbs., w is the distributed load on a 1-ft.

sliver of deck in lb./ft., and L is the spacing between pile caps in ft. Due to the simple beam analysis, the max shear is equal to the reaction forces at both supports. After finding the max shear, the max deflection of the beam was calculated using Equation D10.

$$\Delta_{DL} = \frac{5 \times w \times L^4}{384 \times (E \times 144) \times I_G} \quad (\text{D10})$$

where Δ_{DL} is the max deflection for the distributed load in ft., w is the distributed load on a 1-ft. sliver of deck in lb./ft., L is the spacing between pile caps in ft., E is the modulus of elasticity in lb./in², and I_G is the gross moment of inertia in ft.⁴. The modulus of elasticity is multiplied by 144 in order to convert from in² to ft.². The next load analysis was the point load analysis, which can be seen in Figure D2 below.

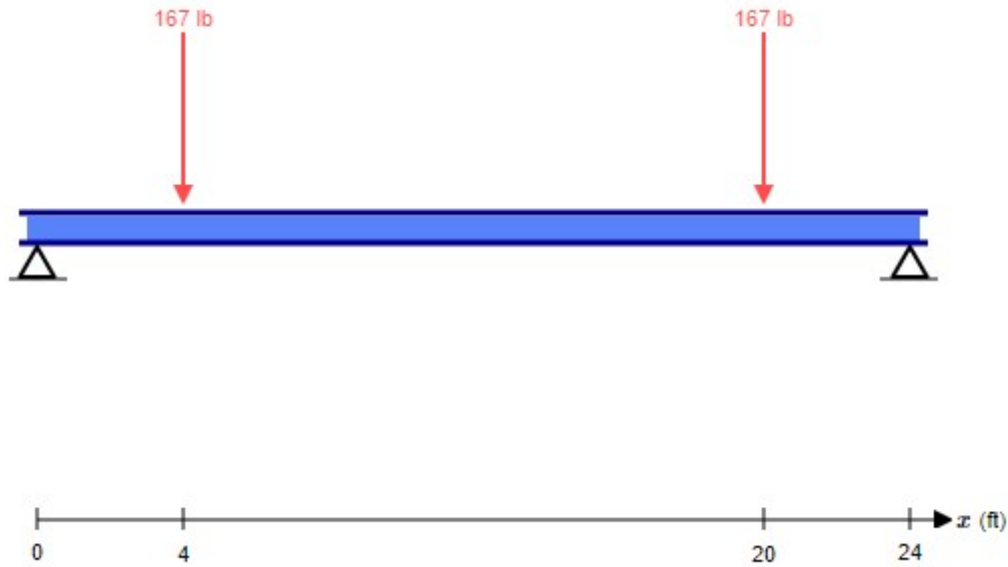


Figure D2: Point Load on Deck (MIDN Russell, 2020)

As previously mentioned, a standard EMS vehicle was the source of the point load on the deck. When positioned on the deck the front and back tires, which were assumed around 16 feet apart, fit between the two pile caps. The distance then from a pile cap to the closest tire was 4 ft. Despite only making contact with the deck in four places, the total weight, around 14,000 lbs., was divided by 42 in order to account for the 42 separate 1-ft. slivers (Weight of an Ambulance, 2020). As seen in the figure above, the weight was then divided by two to simulate two tires encountering the 1-ft. sliver. Each point load weight was found to be about 167 lbs. The max bending moment was calculated using Equation D11.

$$M_{u,PL} = PL \times a \quad (\text{D11})$$

where $M_{u,PL}$ is the max bending moment for the point load in ft.-lbs., PL is the point load in lbs., and a is the distance between a point load and the closest support in ft. Following the moment calculation, the max shear was found using the Equation D12.

$$V_{PL} = PL \quad (\text{D12})$$

where V_{PL} is the max shear for the point load in lbs. and PL is the point load in lbs. Due to the simple beam analysis, the max shear is equal to the reaction forces at both supports. After finding the max shear, the max deflection was calculated using Equation D13.

$$\Delta_{PL} = \frac{PL \times a}{24 \times (E \times 144) \times I_G} \times [(3 \times L^2) - (4 \times a^2)] \quad (\text{D13})$$

where Δ_{PL} is the max deflection for the point load in ft., PL is the point load in lbs., a is the distance between a point load and the closest support in ft., E is the modulus of elasticity in lb/in², I_G is the gross moment of inertia in ft.⁴, and L is the spacing between pile caps in ft. The modulus of elasticity is multiplied by 144 in order to convert from in² to ft.². The final step in the load analysis was combining the distributed load and point load. Due to the simple beam analysis, the max bending moment ($M_{u, max}$), max shear (V_{max}), and max deflection (Δ_{max}) were found by adding each of the components together.

The next step in the structural calculations and final step in the deck analysis was the strength design examination. The following process and equations were found in *Simplified Design of Concrete Structures* by Ambrose and Tripeny. As previously mentioned, the dimensions of the deck and thus the 1-ft. slivers were constant except for the height. Due to using a smaller reference for the analysis, the dimensions were converted from feet to inches. The strength design analysis starts with the establishment of two constants, the concrete compressive strength, f_c' , and the yield strength, f_y . After defining the previous constants, it is assumed that the rectangular structure undergoes a uniformed stress at some percentage of the total compressive strength. This value was calculated using Equation D14.

$$f_{c-A}' = 0.85 \times f_c' \quad (\text{D14})$$

where f_{c-A}' is the allowable compressive strength in lb./in.², 0.85 is the assumed percentage, and f_c' is the compressive strength in lb./in.². The next step after calculating the allowable compressive was finding the nominal moment of inertia using Equation D15.

$$M_n = \frac{M_{u, max}}{\phi} \quad (\text{D15})$$

where M_n is the nominal moment of inertia in ft.-lbs., ϕ is capacity reduction factor, and $M_{u,max}$ is the combined max bending moment from the deck load analysis in ft.-lbs. The capacity reduction factor was another constant established for this analysis. Using Table D2 that was provided from *Simplified Design of Concrete Structures*, three more constants were found by matching their values with the compressive strength and yield strength values. The three new constants were R , a/d , and percentage of reinforcing steel, p .

Table D2: Factors found using f'_c and f_y (*Simplified Design of Concrete Structures*, 2020)

f_y		f'_c		p	a/d	R	
ksi	MPa	ksi	MPa			ksi	kPa
40	276	2	13.8	0.0291	0.685	0.766	5280
		3	20.7	0.0437	0.685	1.149	7920
		4	27.6	0.0582	0.685	1.531	10600
		5	34.5	0.0728	0.685	1.914	13200
60	414	2	13.8	0.0168	0.592	0.708	4890
		3	20.7	0.0252	0.592	1.063	7330
		4	27.6	0.0335	0.592	1.417	9770
		5	34.5	0.0419	0.592	1.771	12200

In order to determine the depth that the reinforcing steel will be placed from the top of the deck, Equation D16 was used.

$$d = \sqrt{\frac{M_n \times 12}{R \times b}} \quad (\text{D16})$$

where d is the depth to reinforcing steel in in., M_n is the nominal moment of inertia in ft.-lbs., R is one of the constants found from Table D2 in lb./in², and b is the width in in. The nominal moment of inertia was multiplied by 12 to convert from ft. to in. The depth value was then compared to a desired depth, which was calculated by subtracting the height by 2 in. The final step for the strength design analysis was to find the area of reinforcing steel using Equation D17.

$$A_s = p \times b \times d \quad (\text{D17})$$

where A_s is the area of reinforcing steel in in.², p is the percentage of reinforcing steel, b is the width of the sliver in in., and d is the depth to reinforcing steel in in. Using Figure D3 below, the number of steel cables needed was found. The area of reinforcing steel was used as a total, thus the individual areas of each cable had to add up to the area of reinforcing steel in the 1-ft. sliver.

Thus, there should be 2, #8 bars per 1-ft. section and 84 bars across the entire deck.

Imperial Bar Size	"Soft" Metric Size	Weight per unit length (lb/ft)	Mass per unit length (kg/m)	Nominal Diameter (in)	Nominal Diameter (mm)	Nominal Area(in ²)	Nominal Area (mm ²)
#3	#10	0.376	0.561	0.375	9.525	0.11	71
#4	#13	0.668	0.996	0.500	12.7	0.2	129
#5	#16	1.043	1.556	0.625	15.875	0.31	200
#6	#19	1.502	2.24	0.750	19.05	0.44	284
#7	#22	2.044	3.049	0.875	22.225	0.6	387
#8	#25	2.67	3.982	1.000	25.4	0.79	509
#9	#29	3.4	5.071	1.128	28.65	1	645
#10	#32	4.303	6.418	1.27	32.26	1.27	819
#11	#36	5.313	7.924	1.41	35.81	1.56	1006
#14	#43	7.65	11.41	1.693	43	2.25	1452
#18	#57	13.6	20.284	2.257	57.33	4	2581

Figure D3: Rebar Sizes (Harris Supply Solutions, 2020)

As seen at the bottom of Table D1, the third trial of the deck analysis or the one with the height at six inches gave us the closest comparison of depth to reinforcing steel and desired depth. Unfortunately, structural engineering codes do not allow for the thickness of a solid rectangular deck to be less than 1/20th of the deck's length. At a value of 1.1 ft., due to 1 ft. of deck slab resting on the pile cap, the successful height of 6 in. was exceeded. Fortunately, by changing the design from a solid deck to a ribbed deck, the six in. height could be used.

The next step in the structural calculations was the pile cap load analysis. The consistent parameters were the same as those in the deck analysis and can be found in Table D3 as well as the calculated values for the pile cap analysis.

Table D3: Pile Cap Analysis (MIDN Russell, 2020)

	Trial 1	Trial 2	Trial 3
Pile Cap Analysis			
γ_c (lb/ft ³)	150	150	150
Design Live Load (lb/ft ²)	100	100	100
Modulus of Elasticity, E (lb/in ²)	6.00E+06	6.00E+06	6.00E+06
Gross Moment of Inertia, I_G (ft ⁴)	1.666667	0.666667	0.41
Deck Calculations			
Width (ft)	42	42	42
t (ft)	0.5	0.5	0.5
Span btw pile caps (ft)	24	24	24
A_{deck} (ft ²)	1008	1008	1008
V_{deck} (ft ³)	504	504	504
Deck Weight, DL (lbs)	75600	75600	75600
Deck Live Load, LL (lbs)	100800	100800	100800
Design Deck Load, Q_d (lbs)	252000	252000	252000
Pile Cap			
Thickness, h (ft)	2	2	2
Length, L_e (ft)	42	42	42
Width, b (ft)	2.5	1.00	0.62
Pile Cap Weight, DL_c (lb)	31500	12600	7749
Design Cap Weight, Q_c (lb)	37800	15120	9298.8
Analysis			
Distributed Load on Pile Cap, w (lb/ft)	6900	6360	6221
Spacing btw piles, L (ft)	21	21	21
Max Bending Moment, $M_u = M_1$ (ft-lbs)	380362.5	350595	342954.7
Max Shear, V (lb)	90562.5	83475	81655.88
Pile Reactions, $R_1 = R_3$ (lb)	54337.5	50085	48993.53
Pile Reactions, R_2 (lb)	181125	166950	163311.8
Max Deflection, Δ (ft)	0.005037	0.011608	0.018463
Strength Design			
Thickness= height, h (in)	24	24	24
Width, b (in)	30	12	7
Concrete compressive strength, f_c' (lb/in ²)	4000	4000	4000
Allowable compressive strength, f_{cA}' (lb/in ²)	3400	3400	3400
Yield strength, f_y (lb/in ²)	60000	60000	60000
Capacity reduction factor, ϕ	0.9	0.9	0.9
Nominal Moment of Inertia, M_n (ft-lbs)	422625	389550	381060.8
Factors from Table:			
R-factor (psi)	1417	1417	1417
a/d-factor	0.592	0.592	0.592
Percentage of Reinforcing Steel, p	0.0335	0.0335	0.0335
Use R-factor to solve for d			
Depth to reinforcing steel, d (in)	10.92	16.58	20.91
Desired d in order to achieve $d = h - 3$ in.	21	21	21
Use p-factor to find A_s			
Area of reinforcing steel, A_s (in ²)	10.98	6.67	5.17
Practical Concerns			
Diameter of Piles (in)	30		
Pile Cap Width needed to support piles (in)	36		
Pile Cap Width needed to support deck (in)	30		
Ideal pile cap width for strength design (in)	7		
Area of reinforcing steel for ideal pile cap (in ²)	5.17		
Scaled Pile Cap Width (in)	35.00		
Scaled A_s (in ²) to match pile cap width	24.52		

The gross moment of inertia for the pile cap was found using Equation D1, but the instead of the height and width of the deck being used it was the thickness or height, h , and width, b , of the pile cap. Following the establishment of the consistent parameters, the deck dimensions, area, volume, dead load, live load, and design deck load were defined and calculated. Due to the completion of the deck analysis, the thickness of the deck was known and, similar to the other two dimensions, was kept constant throughout the three trials. Using Equations D2 through D6, the rest of the values for the deck were found. After calculating the parameters of the deck, dimensions of the pile cap were established. As previously mentioned the height, h , and length, Le , of the pile cap remained the same while the width, b , changed during the three trials. Using the dimensions and Equation D18, the pile cap weight was found.

$$DL_c = h \times Le \times b \quad (D18)$$

where DL_c is the pile cap weight (dead load) in lbs., h is the height of the pile cap in ft., Le is the length of the pile cap in ft., and b is the width of the pile cap in ft. After finding the pile cap weight, the design cap weight was calculated using Equation D19.

$$Q_c = 1.2 \times DL_c \quad (D19)$$

where Q_c is the design cap weight in lbs., 1.2 is the safety factor for dead loads, and DL_c is the pile cap weight in lbs. Due to the pile cap position's under the deck, there was not a live load that affected it. However, the dead and live loads for the deck were incorporated in the distributed load analysis for the pile cap, seen in Figure D4 below.

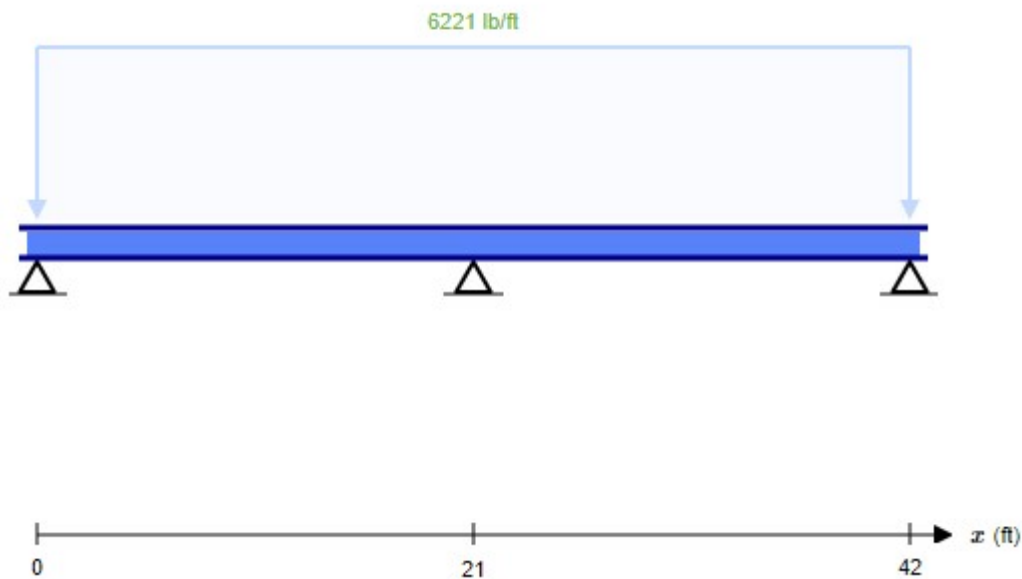


Figure D4: Distributed Load on Pile Cap (MIDN Russell, 2020)

Unlike the deck load analysis, the pile cap load analysis had three supports, from the three piles, and only examined a distributed load. The piles are spaced at a length of 21 ft. apart from each other, thus two spaces between three piles to fill in 42 ft. Using Equation D20, the distributed load was calculated.

$$w = \frac{Q_d + Q_c}{Le} \quad (\text{D20})$$

where w is the distributed load on the pile cap in lb./ft., Q_d is the design deck load in lbs., Q_c is the design cap weight in lbs., and Le is the length of the pile cap in ft. The max bending moment of the pile cap was found using Equation D21.

$$M_u = \frac{w \times L^2}{8} \quad (\text{D21})$$

where M_u is the max bending moment in ft.-lbs., w is the distributed load of the pile cap in lb./ft., and L is the spacing between piles in ft. After finding the max bending moment, the max shear was calculated using Equation D22.

$$V = \frac{5 \times w \times L}{8} \quad (\text{D22})$$

where V is the max shear in lbs., w is the distributed load of the pile cap in lb./ft., and L is the spacing between piles in ft. The pile reactions in the two outside piles are the same and found using Equation D23.

$$R_1 = R_3 = \frac{3 \times w \times L}{8} \quad (\text{D23})$$

where R_1 and R_3 are the reaction forces in lbs. for the outside piles, w is the distributed load of the pile cap in lb./ft., and L is the spacing between piles in ft. The pile reaction force for the middle pile was calculated using Equation D24.

$$R_2 = \frac{10 \times w \times L}{8} \quad (\text{D24})$$

where R_2 is the reaction force in lbs. for the middle pile, w is the distributed load of the pile cap in lb./ft., and L is the spacing between piles in ft. The max deflection was found using equation D25.

$$\Delta = \frac{w \times L^4}{185 \times (E \times 144) \times I_G} \quad (\text{D25})$$

where Δ is the max deflection in ft., w is the distributed load of the pile cap in lb./ft., L is the spacing between piles in ft., E is the modulus of elasticity in lb./in², and I_G is the gross moment of inertia in ft.⁴, and L is the spacing between pile caps in ft. The modulus of elasticity is

multiplied by 144 in order to convert from in² to ft.².

The final step in the structural calculations for the deck and the pile cap was the strength design analysis for the pile cap. Fortunately, the process is very similar to the strength design analysis of the deck. All of the constants such as the compressive strength f_c' , yield strength, f_y , the capacity reduction factor, Φ , and the factors from Table D2, stayed the same. Equations D14 through D17 stayed the same with the only difference being that in D15 M_u of the pile cap was used instead of $M_{u, max}$ of the pile cap. The biggest differences in the strength design analysis was that the width changed instead of the height and the desired depth was subtracting three inches from the height instead of two. As previously mentioned, the pile cap width that was successful in almost matching the desired depth for reinforcing steel was 7 in. and yielded a reinforcing steel area of 5.17 in.². This narrow width would not provide enough space for the ends of decks to rest on and it definitely would not cover the 2.5 ft. wide piles. In order to correct this problem the width and reinforcing steel area were increased by a factor of 4.743 which resulted in a scaled pile cap width of 35 in. and reinforcing steel area of 24.52 in.². Using Figure D3, the number of steel cables was determined to be 11 #14 bars.

Appendix E

In order to prove the concept of the alternative Pier being designed, a structural analysis of the Piles has been conducted. Using chapter 10, Pile Foundations, of Soils and Foundations, 3rd edition (Liu and Evett, 1992) the pile analysis was conducted with the help and recommendations of Professor David Kriebel, USNA. The selected alternative design has three piles per bent as opposed to the current pier's five.

Deck and Pile Cap Load

Considering this, each bent must structurally support a concrete deck 42 ft. long (l) and 24 ft. wide (w). Additionally each bent of three piles must be connected by a pile cap. From structural calculations carried out in Appendix D, the alternative design's deck was 0.5 ft. thick (t). With the aforementioned dimensions and a known cubic weight of concrete (γ_c) of 150 lb./ft.³, using Equation E1 the deck dead load (DL) was found. Using the dimensions of the pile cap, its dead load was also found using the same equation.

$$DL = \gamma_c * l * w * t \quad (E1)$$

Only the deck had a live load (LL) and this was found using a live pressure constant of (σ_{live}) 100 lb/ft.² and the dimensions of the deck. The equation is located below.

$$LL = \sigma_{live} * l * w \quad (E2)$$

From Appendix D, the loads had to be factored together for a total load bearing down on the piles. The factored load equation (Q_u) is shown in Equation E3: 1.2 is the safety factor for dead loads and 1.6 is the safety factor for live loads.

$$Q_u = 1.4DDL + 1.6DLL \quad (E3)$$

The findings from Equations E1, E2, and E3 are summarized in the table below.

Table E1: Breakwater Decision Matrix (MIDN Myers, 2020)

Deck DL (lb)	75600
Deck LL (lb)	100800
Pile Cap DL (lb)	31500
Pile Cap LL (lb)	0
Factored Deck Load (Qu)	312480

With the chosen deck geometry the loads of the pile cap and deck do not evenly disperse between each pile. As shown in Figure E1 below, the middle pile must be able to support 50% of the load (Kriebel, 2020). In the alternative design the piles for the alternative pier would be mass produced. Since each pile would be the same length, shape, and material in this scenario, a well-engineered design must model every pile off of the pile which absorbs most of the load: in this case the middle pile. By designing for the maximum load there is no chance that any pile can fail under load.

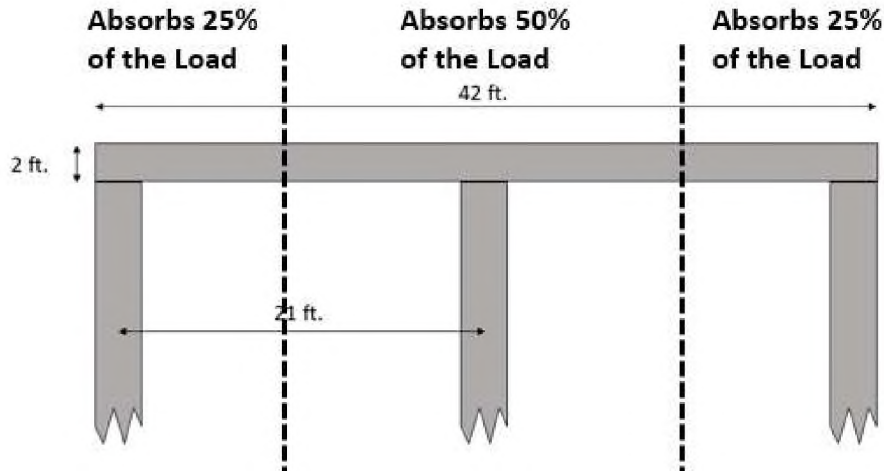


Figure E1: Pile Load Dispersion (MIDN Myers, 2020)

Equation E4 calculates the load per pile which is half of the factored load (Q_u).

$$\text{Load per Pile} = 0.5 * Q_u \quad (\text{E4})$$

Design Constants

Before moving on the next step in the pile design process, the below constants were used for circular and square. These constants were found in the Soils and Foundations textbook. For notation purposes the table contains: the density of seawater (γ_w), the density of submerged sand (γ_{sub}), and the coefficient of friction between sand and concrete ($\tan f$), the distance from the water surface to the seafloor (d), the circumference (C) for a circular pile, and the perimeter of a square pile (P). Some constants had to be found using API design guidelines, for example, K is the API constant for the coefficient of lateral earth pressure to convert vertical pressure (P_v) to horizontal pressure acting inward on a pile. The K of 0.8 assumes the pile is in compression and is driven into undisturbed sand sediment (Liu and Evett, 1992).

Table E2: Circular pile design constants (MIDN Myers, 2020)

Circular Piles				
Diameter, B (ft)	1.5	2	2.5	3
γ_w (lb/ft ³)	64	64	64	64
γ_{sub} (lb/ft ³)	60	60	60	60
$\tan f$	0.45	0.45	0.45	0.45
K	0.8	0.8	0.8	0.8
d (water depth)	30	30	30	30
D_c (critical Length)	15	20	25	30
C (circumference)	4.7	6.3	7.9	9.4

Table E3: Square pile design constants (MIDN Myers, 2020)

Square Piles				
Base Length, B (ft)	1.5	2	2.5	3
γ_w (lb/ft ³)	64	64	64	64
γ_{sub} (lb/ft ³)	60	60	60	60
$\tan\phi$	0.45	0.45	0.45	0.45
K	0.8	0.8	0.8	0.8
d (water depth)	30	30	30	30
D_c (critical Length)	15	20	25	30
P (perimeter)	6	8	10	12

The variance in the pile tip area for the circular and square piles differentiated the load bearing capacity for each pile shape causing the square piles to have a higher tip capacity. Similarly, the variance in pile wall surface area for each pile shape caused the frictional bearing capacity to be greater for the square piles. In these equations L is how deep the pile is driven into the seafloor.

$$A_{tip,circular} = .25 * \pi B^2 \quad (E5)$$

$$A_{tip,square} = B^2 \quad (E6)$$

$$A_{surface,circular} = \pi BL \quad (E7)$$

$$A_{surface,square} = 4BL \quad (E8)$$

The critical depth (D_c) in Tables E2 and E3 comes from a study that has shown that P_v does not keep increasing with depth. At the critical depth the bearing pressure on the walls of the piles becomes constant.

$$D_c = 10 * B \quad (E9)$$

The below graph from page 294 of Soils and Foundations (Liu and Evett, 1992) shows the relationship between depth and pressure on the pile.

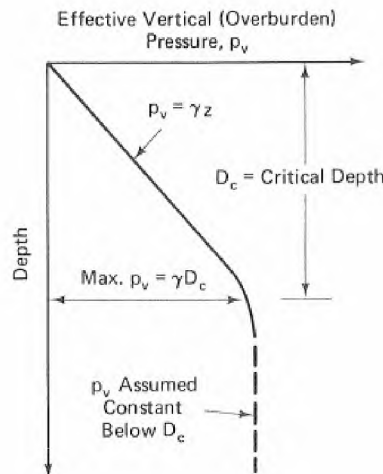


Figure E2: Pile depth and pressure relationship (Liu and Evett, 1992)

Tip Resistance

The tip resistance (Q_{tip}) was calculated using the pile tip area, the pressure at the tip (P_v), and the bearing capacity factor for typical sand (N_q) with a friction angle of 30 degrees (Soils and Foundations page 244) of 20 from the figure below.

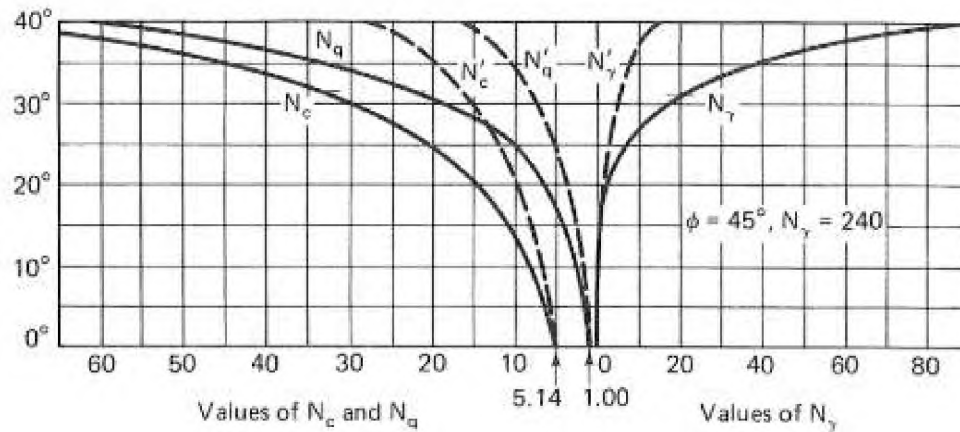


Figure E2: Pile depth and pressure relationship (Liu and Evett, 1992)

$$P_v = N_q * D_c \quad (E10)$$

$$Q_{tip} = N_q * P_v * A_{tip} \quad (E11)$$

Data calculated for the alternative design piles summarized in the tables below.

Table E4: Circular pile tip resistance (MIDN Myers, 2020)

Circular Piles				
Diameter, B (ft)	1.5	2	2.5	3
A _{tip}	1.8	3.1	4.9	7.1
P _v tip	900	1200	1500	1800
N _q	20	20	20	20
Q _{tip} (lbs)	31808.6	75398.22	147262	254469

Table E5: Square pile tip resistance (MIDN Myers, 2020)

Square Piles				
Base Length, B (ft)	1.5	2	2.5	3
A _{tip}	2.3	4.0	6.3	9.0
P _v tip	900	1200	1500	1800
N _q	20	20	20	20
Q _{tip} (lbs)	40500	96000	187500	324000

Frictional Resistance

The frictional resistance (Q_f) of the pile is determined in large by the unit value of skin friction (f). It is an average value over the driven length of the pile (L).

$$f = K * \tan f * \gamma_{sub} \quad (E12)$$

From Soils and Foundations (Eq. 10-2), frictional resistance can be expressed as the following equation.

$$Q_f = f * A_{surface} \quad (E13)$$

However since it is known that pressure becomes constant after the critical depth, from Figure E2, $A_{surface}$ can be algebraically rearranged to input the driven pile length (L) into Equation E13.

$$Q_f = f * L * C \quad (E14)$$

This relationship can be further targeted by multiplying the piles circumference by the area under the P_v vs. Depth curve. The figure below shows the areas under the curve.

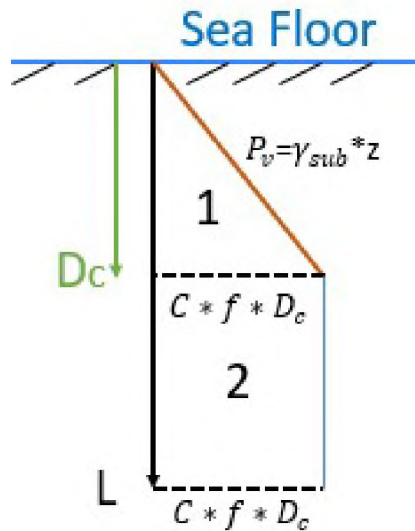


Figure E3: Pile depth and pressure areas 1 and 2 (Liu and Evett, 1992)

$$Q_f = \text{area under plot of } f * C \quad (E14)$$

Area #1 from Figure E3 came from the following area equation for a triangle.

$$Q_{f\#1} = 0.5 * D_c * C * f * D_c \quad (E15)$$

Area #2 from Figure E3 came from the following area equation for a rectangle.

$$Q_{f\#2} = (L - D_c) * C * f * D_c \quad (E16)$$

The total frictional resistance is then equal to the sum of Equations E15 and E16. With substitution and simplifying it ended up being the following.

$$Q_f = (f(0.5 * D_c^2)) + ((L - D_c)D_c) \quad (E17)$$

Equation 17 allowed various L 's to be plugged in to systematically find the total resistance of the pile.

Net Weight per Pile

The load per pile is located earlier in the appendix in Equation E4. The weight of the pile multiplies the density of concrete by the volume of the pile. The volume came from the height from the water level to the pile cap (15), the depth of the water (d), and the driven length (L).

$$Weight\ of\ Pile = \gamma_c * A_{tip} * (15 + L + d) \quad (E18)$$

The buoyancy of the pile was a product of the submerged volume times the density of seawater (γ_w).

$$Pile\ Buoyancy = \gamma_w * A_{tip} * (L + d) \quad (E19)$$

The net load per pile was a combination of the load per pile, weight of the pile, and buoyancy of the pile. It is the result of the gravitational forces and buoyant forces acting on the pile.

$$Net\ Load\ per\ Pile = Load\ per\ Pile + Weigh\ of\ Pile - Pile\ Buoyancy \quad (E20)$$

The design load (Q_{design}) that the pile must support was based on the net load per pile. In order to engineer the piles with safety mind, the net load per pile was multiplied by a factor of safety ($F.S.$) of 2. It is common practice to use a factor of safety of 2 when when reliable materials are being used and environmental conditions are not severe (Engineering ToolBox, 2010). These piles have been designed with safety in mind.

$$Q_{design} = F.S.* Net\ Load\ per\ Pile \quad (E21)$$

Total Pile Resistance

A piles total resistance to load was the sum of the tip and frictional resistance from Equation E11 and E17.

$$Total\ Resistance = Q_{tip} + Q_f \quad (E22)$$

Solving for Pile Length

The pile length required was found by using excel and creating a spread sheet for various circular and square pile diameters. The only unknown was the driven length (L) that each diameter would require. However, it was known that the piles total resistance from Equation E22 had to be equal to or greater than the piles design load (Q_{design}) from Equation E21 for it to be considered a safe design.

$$Q_{design} \leq Q_{tip} + Q_f \quad (E23)$$

Results

The tables previously included in this appendix had values that were essential to calculating results for each pile diameter, but they did not need driven length (L) to be calculated. The creation of the following table followed the procedure that has been laid out in the appendix, however, it will only include the pertinent final results that were calculated. Using excel different Driven Lengths (L) were plugged into the spreadsheet until the total pile resistance was equal to or just greater than the pile design load.

Table E6: Circular pile results (MIDN Myers, 2020)

Circular Piles				
Diameter, B (ft)	1.5	2	2.5	3
Driven Length (ft)	253	136	79	45
Total Pile Length (ft)	298	181	124	90
Total Volume (ft ³)	527	569	609	636
Load per Pile (lb)	156240	156240	156240	156240
Weight of pile (lb)	78991	85294	91303	95426
Buoyancy of pile (lb)	32007	33376	34243	33929
Net load per pile (lb)	203225	208158	213299	217737
Q _{design} (lb)	406450	416316	426598	435473
Q _{tip} (lb)	31809	75398	147262	254469
Q _f (lb)	374833	342006	282036	183218
Total Resistance (lb)	406641	417405	429299	437687

Table E7: Square pile results (MIDN Myers, 2020)

Square Piles				
Base Length, B (ft)	1.5	2	2.5	3
Driven Length (ft)	199	105	59	31
Total Pile Length (ft)	244	150	104	76
Total Volume (ft ³)	549	600	650	684
Load per Pile (lb)	156240	156240	156240	156240
Weight of pile (lb)	82350	90000	97500	102600
Buoyancy of pile (lb)	32976	34560	35600	35136
Net load per pile (lb)	205614	211680	218140	223704
Q _{design} (lb)	411228	423360	436280	447408
Q _{tip} (lb)	40500	96000	187500	324000
Q _f (lb)	372276	328320	251100	124416
Total Resistance (lb)	412776	424320	438600	448416

In complying with safe design practices, the total pile resistance had to be larger than the design load to solve for the driven length. The results show that the circular piles must typically be driven deeper into the seafloor, but use less concrete material than similarly sized square piles. Since the square piles were found to require more total volume they were ruled out for the alternative design in favor of the circular piles. Computing the required driven length (L) showed that the piles smaller than 2.5 ft. in diameter must be extremely long at 150 ft. and 255 ft. in order handle the load. Nothing above 120 ft. in length would be practical for installation. While the 3 ft. circular pile requires shorter total length than the 2.5 ft. diameter pile, the total volume of the 2.5 ft pile is smaller and would therefore be cheaper to manufacture. Minimizing cost if possible has been emphasized as a design goal and the 2.5 ft. diameter pile has a more economical total volume. For these reasons the 2.5 ft. diameter, 124 ft. long circular piles were used for the alternative design.

Appendix F

The following processes and equations were used to calculate the theoretical wave transmission and wave forces that the alternative breakwater design would have and be impacted with.

Wave Transmission

The equation to find the wave transmission can be seen in Equation F1 below. This was used in order to show how the waves would act after they had passed under the wave barrier. When calculating wave transmission, the smaller the number, the better the transmission is.

$$K_{Tmo} = e^{-15\left(\frac{w}{L_p}\right)^{1.4}} \quad (\text{F1})$$

In this equation, w is the draft of the wave barrier and is the height of the wave barrier that the water covers. Then, L_p is the wavelength associated with the peak wave period.

Wave Forces

The wave forces were calculated by using three different equations. The first equation that was used was the wave force for the full depth of the water. The equation can be seen below,

$$F_{Wmo} = b\rho g H_{mo} \frac{1}{K_p} \tanh(k_p d) \quad (\text{F2})$$

where in this equation, b , is the board width, ρ , is the density of seawater, g , is the acceleration of gravity, k_p , is the peak wave period, and H_{mo} , is the average wave height. Once the force of the wave for the full depth has been found then the next equation can be applied. This equation is for the partial depth wave force, and it can be seen below.

$$F_{mo} = F_{Wmo} \left(\frac{w}{d}\right)^{0.386} \left(\frac{d}{L_p}\right)^{-0.7} \quad (\text{F3})$$

where all the variables represent the same values as those in Equation F1. After finding the partial depth forces, then the design force could be found. The design force equation can be seen below.

$$F_{design} = 1.8 F_{mo} \quad (\text{F4})$$