

5.0 Concept Design

5.1 General Improvements

a) Accessibility/ADA

Portions of the existing sidewalk were in compliance, however there was a considerable length that was non-conforming. Any work in this area should bring the sidewalk up to the minimum required conditions (2% max. cross slope & 5% max. running slope). The three feasible options for this predicament are as follows:

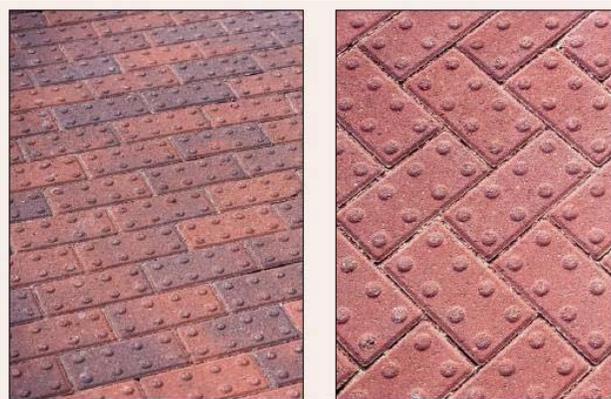
- Option one: lower the sidewalk. Starting at the street level, establish the 8" curb height and add the conforming slope upward until the curb and barrier at the face of the seawall is reached. This will necessitate a much higher curb, in order to maintain the top of seawall elevation. In the worst case the curb height would be approximately 2'-6" above the sidewalk surface.
- Option two: Raise the sidewalk on the street side however this option would create an obstruction for car doors and impair the parking lane. This option would also require a barrier rail on the landward side of the walkway, increasing installed cost.
- Option three: A hybrid, maintain a minimum walkway width of 36" and then slope down to the curb with a concrete or other impervious surface. This option would also require a barrier rail on the landward side of the walkway, increasing cost and maintenance.

Although not explicitly required in the current ADA Standards for Accessible Design, it is recommended that additional curb ramps and crosswalks be installed along Murray Blvd. at several of the intersections. It should also be noted that if these curb ramps and crosswalks are installed, compliance with the curb ramp on the other side of Murray Blvd. will also be needed.

Current ADA standards for ramps require a maximum running slope of 1:12 (8.33%), a maximum cross slope of 1:48 (2.00%) and a minimum ramp width of 36" (although 60" is recommended). In addition, each ramp shall have level (2% or less in all directions) landing areas at the top of the ramp which measure at least 60" long and shall be at least as wide as the ramp. It should also be noted that the adjoining surface of the roadway to the ramp shall have a maximum slope of 1:20 (5.00%).

Detectable warning surfaces are also required at all ramp landings and shall consist of truncated domes 24" wide spanning the entire width of the ramp. There is no specific material type or color required for these surfaces and there are options, such as brick pavers, which will help to provide a more aesthetically pleasing ramp. Detectable warning surfaces shall contrast visually with adjacent walking surfaces either light-on-dark, or dark-on-light.

Although not explicitly required by the ADA requirements, crosswalk lines shall be required at all crossings on SCDOT maintained roads. In



this case, all designated pedestrian crossing points along Murray Blvd. shall be marked with 8" solid white lines, not less than 6' apart.

ADA does not dictate requirements for the adjacent barrier / railing along this corridor. However, should the running slope exceed 5% at any point along this corridor, this area will be considered a ramp and will require a handrail to be in compliance with the current ADA Standards.

b) Sea Level rise Accommodation

Coastal South Carolina, specifically Charleston, is particularly vulnerable to elevated water levels due to its low-lying geography. The current and future effects of sea level rise on the Atlantic Coastline is a concern. The battery wall surrounding the downtown Charleston may not be able to withstand rising sea levels.

As with many coastal areas, sea level rise and land subsidence threats are increasingly more dangerous every year. It has been determined by the research and investigation from several major groups including National Oceanic and Atmospheric Administration (NOAA), South Carolina Sea Grant Consortium (SCSGC), Social and Environmental Research Institute (SERI), etc., that the sea level can rise for a number of well-known reasons. This includes an increase in volume of water from the expansion of the warmer water and the melting of glaciers and ice sheets interjecting more water into seas. Based on predictions¹ and observations from NOAA Station 8670870 at Fort Pulaski, sea level has risen an average of 0.12 inches per year (1.2 in/decade or 1.0 ft./century) in the Southeast Atlantic since 1935 when the station was installed.

Using the Sea-Level Change Calculator (v2014.88) provided by the US Army Corps of Engineers (USACE), a site specific forecast was developed for the Charleston harbor. Based on a 100 year design life for coastal structures, the historically based, predicted sea level rise for the Charleston harbor over that time period would increase mean sea level by approximately 12 inches by 2115.

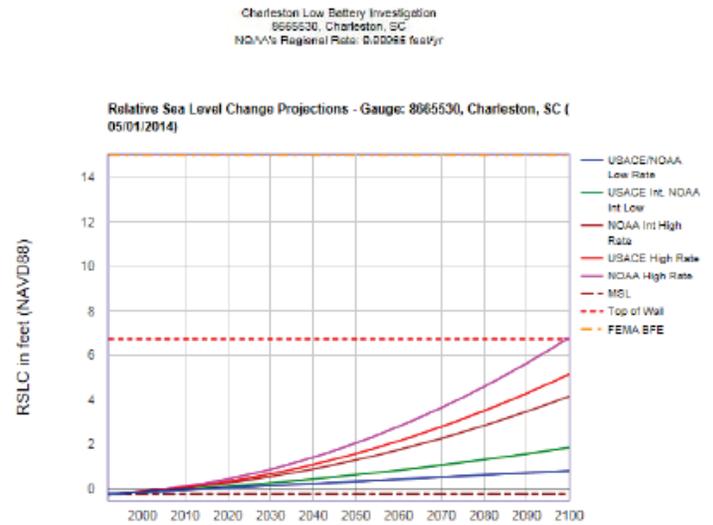
The following chart and graph show pertinent elevations and alternate sea-level change predictions. It is important to note that the elevations shown below are according to the North American Vertical Datum (NAVD88) were the remainder of the elevations listed in this report are National Geodetic Vertical Datum (NGVD29) as requested by the client.

¹ South Carolina Sea Grant Consortium. 2014. "Sea Level Rise Adaptation Report Beaufort County, South Carolina." <http://www.scseagrant.org/Content/?cid=251>



Charleston Low Battery Investigation
866530, Charleston, SC
NOAA's Regional Rate: 0.00066 feet/yr
All values are expressed in feet relative to NAVD88

Year	USACE Low NOAA Low	USACE Int NOAA Int Low	NOAA Int High	USACE High	NOAA High
1992	-0.22	-0.22	-0.22	-0.22	-0.22
1995	-0.19	-0.19	-0.19	-0.19	-0.19
2000	-0.14	-0.14	-0.13	-0.12	-0.11
2005	-0.10	-0.08	-0.05	-0.03	-0.01
2010	-0.05	-0.02	0.05	0.07	0.12
2015	0.00	0.05	0.15	0.20	0.27
2020	0.05	0.12	0.27	0.34	0.45
2025	0.10	0.20	0.41	0.50	0.66
2030	0.15	0.28	0.58	0.68	0.88
2035	0.20	0.36	0.72	0.88	1.14
2040	0.24	0.45	0.90	1.10	1.42
2045	0.29	0.54	1.09	1.33	1.73
2050	0.34	0.64	1.30	1.60	2.06
2055	0.39	0.74	1.52	1.86	2.42
2060	0.44	0.85	1.76	2.15	2.80
2065	0.48	0.96	2.01	2.46	3.21
2070	0.53	1.07	2.27	2.79	3.64
2075	0.58	1.19	2.55	3.14	4.10
2080	0.63	1.32	2.84	3.50	4.59
2085	0.68	1.45	3.15	3.88	5.10
2090	0.73	1.58	3.47	4.29	5.63
2095	0.77	1.72	3.81	4.71	6.19
2100	0.82	1.88	4.18	5.15	6.76

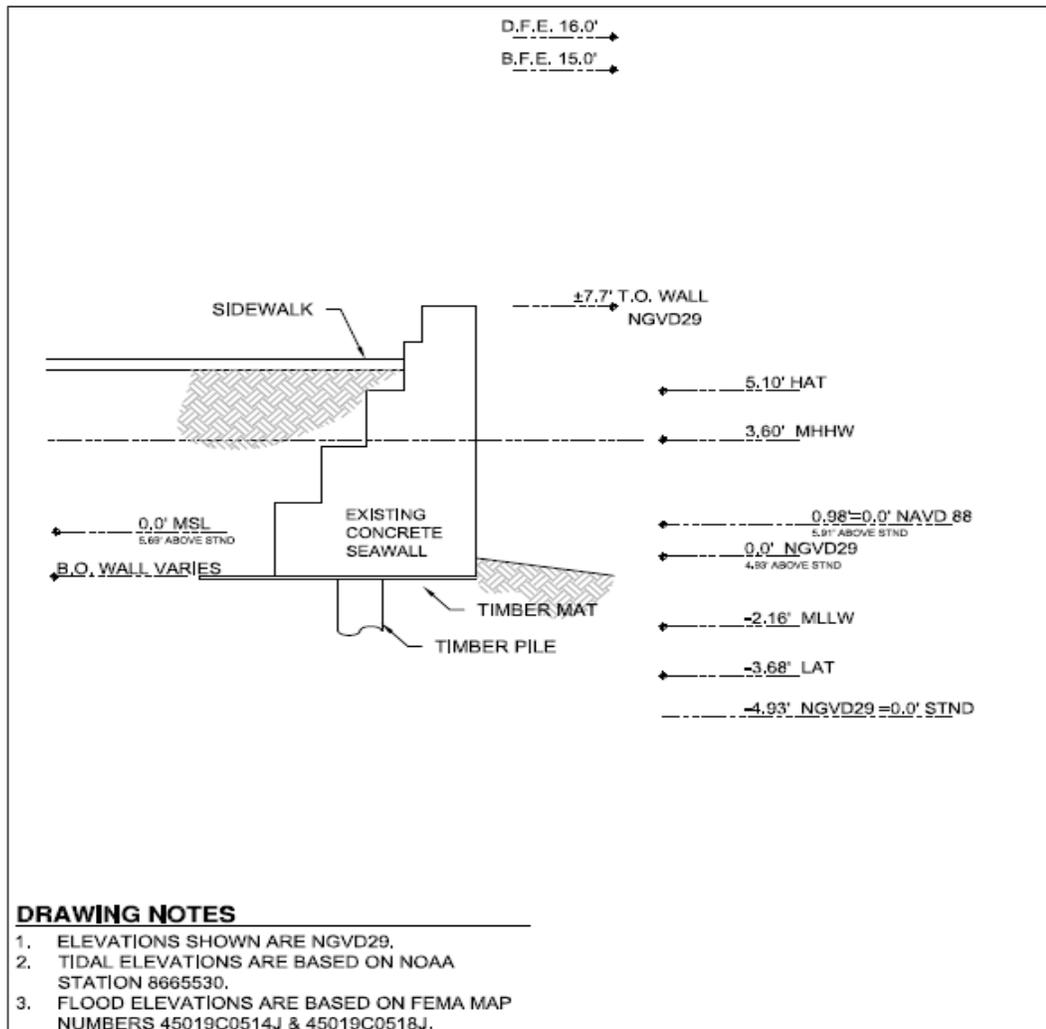


The graph above shows the Federal Emergency Management Agency (FEMA) based Base Flood Elevation (BFE) in an orange horizontal dashed line, obtained from Flood Maps for the area. The horizontal dashed red line represents the approximate top of existing seawall, while the three curves are alternative predictions, high, intermediate and low. The blue line, the low curve, represents the historic rate of sea level change. The green line is the intermediate curve and is a theoretical prediction that is developed using projections from the National Research Council and USACE calculations. The solid red line, or high curve, is again a theoretical prediction considering extreme events, such as the rapid loss of ice from Antarctica and Greenland.

The city of Charleston has set a sea level rise level at 2.5' in 50 years for planning purposes. Using the baseline of sea level rise, would raise the mean higher high water to 6.1' NGVD29 which is 1.6 feet below the top of wall. And the highest atmospheric tide at 7.60' which is right at the top of the wall. Based on this, an increase in the top elevation of wall should be considered.

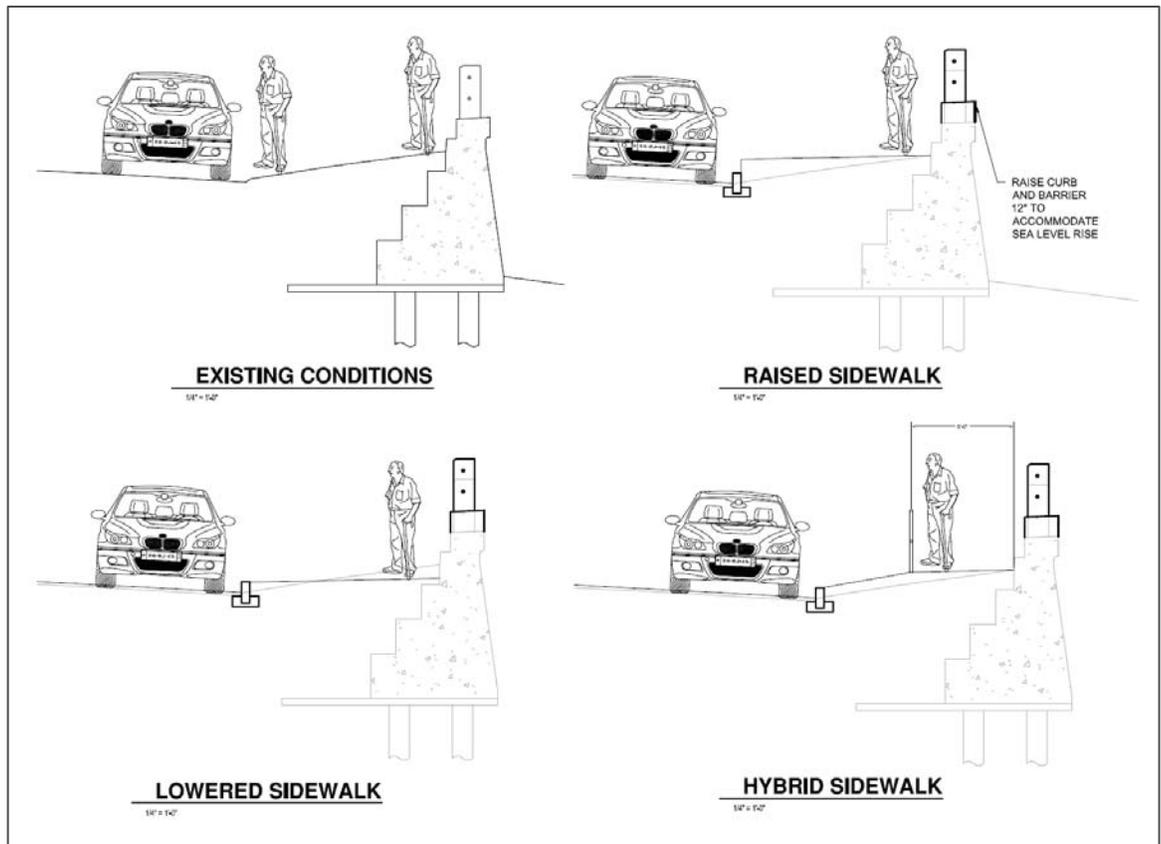
Aside from sea level rise, consideration should be given for raising the wall a modest height to protect against overtopping during storm events such as seen during the 1,000 year storm of October, 2015. Since overtopping of the wall was frequently noted during this unprecedented event, an increase of at least 1 foot is reasonable to aid in defending against future storms and this will be further investigated during final design.

It is important to note that the purpose of the low battery seawall is not to prevent overtopping during a storm event. As shown below the FEMA predicted base flood elevation (BFE) greatly exceeds the height of the wall. The wall is to prevent the normal water levels from overtopping.



The sketch above shows the existing sea level conditions of the seawall at the Low Battery.





Sidewalk and curb options

c) Seismic Resiliency

Resilience is defined in the dictionary as the ability to become strong, healthy or successful again after something bad happens. It also refers to the ability of something to return to its original shape after being deformed in some fashion. The objective of seismic resiliency is to minimize detrimental impacts to a community's infrastructure after an earthquake event and to increase a community's ability to return to a pre-event way of life. This discussion focuses on potential improvements to improve the low battery wall's resiliency and improve its ability to perform during and after an earthquake.

As the wall stands today, it was not designed to perform under the expected seismic loads indicated in present day codes. This can be said of most historical and pre-seismic code buildings, bridges and other infrastructure. In recent years, both the historic Dock Street Theater and Charleston City Hall have undergone structural repairs that included some level of seismic improvements while the renovation work was taking place. If the wall were in good condition, and no rehabilitation activity was being contemplated, it would most likely not undergo a seismic upgrade. Realistically, it could prove difficult, or cost prohibitive, to achieve a modern day compliant seismic design without totally rebuilding the wall. However, since significant structural improvements are being planned, consideration should be given to improving the seismic resiliency of the wall in conjunction with the upcoming strengthening work. While not assuring that the wall would sustain no damage, improvements could be designed into the rehabilitation

to minimize damage and make whatever damage occurs more repairable. The importance of this consideration is underlined by the dependence of the neighborhood adjacent to Murray Boulevard upon the wall to hold back the tidal and storm induced floodwaters of the Ashley River.

Since the probable rehabilitation work is most likely going to be comprised of foundation and wall strengthening vs. a complete rebuild, a seismic resiliency improvement strategy appears to be the most applicable route. Our measure of effectiveness for achieving seismic resiliency of the wall would be to answer “yes” to the following:

1. Have we reduce the wall's probability of total failure?
2. Have we reduced the consequences and collateral damage that would occur as a result of any partial failures?
3. Have we reduced the time required to repair the wall to its pre-event functionality?

Through proper modeling and analysis during the design phase, our goal will be to answer “yes” to the three questions above and to quantify a level of improvement in the wall's lateral capacity as well as the ability to resist settlements due to liquefiable soils.

5.2 Alternatives

Based on our investigation the options available for repair, renovation or replacement are as follows:

a) Alternative 1 – Perform Maintenance Activities with no structural improvements

The lowest cost and least recommended alternative is to not perform structural improvements to increase the wall's capacity, but to only perform maintenance activities aimed at arresting the wall and foundation's further deterioration. The City already is providing maintenance for the sidewalk and railing areas as well as isolated settlement addressment. These maintenance costs will continue to escalate as the cracks in the wall widen allowing soil to continue to migrate from behind the wall with the penetrating tide and storm water runoff. The timber pile connections to the underside of the wall will eventually fail due to the decay that will proceed unchecked at an exponential rate. This will leave the wall relying on soil bearing only for lateral, and in some instances, vertical support. Based on observations of the wall and its deterioration to date, it is unlikely (although still possible) that the wall would suffer a sudden & catastrophic failure under ordinary use and typical loading conditions. The gradual weakening generally means a serious failure would likely occur during a storm or severe hazard event like an earthquake, when the City is least likely to have the resources available to effect repairs.

b) Alternative 2 – Complete Reconstruction

Full replacement of the wall, similar to the previously completed work performed at "the turn", is a viable option to ensure that the wall is replaced with high quality material and built to last for an approximately 75 year service life, or more. This option considers complete demolition and replacement and is the most expensive, time consuming, and disruptive to the resident's quality of life and to the tourism industry in the area. Because of the recent work undertaken by the City at "the turn", this alternative is also the easiest to quantify the costs, time and permitting requirements for.

c) Alternative 3 – Rehabilitation & Underpinning Options

A structural up fit to the wall and its foundations through a targeted reconstruction is an alternative we believe has merit to achieve the desired results of:

1. Preventing further settlement of both the wall, sidewalk and the fill being retained.
2. Sealing and strengthening the wall face from further migration of fill through the cracks by not allowing the tidal and storm water infiltration that has been occurring.
3. Improving the lateral resistance of the wall to prevent rotation under normal and extreme hazard loads.
4. Increase the seismic resiliency of the overall wall system.

A structural, "above the footing level", upfit could entail demolition of the curb and railing system in place and replacing it with a similar profile, a reinforced concrete curb and concrete posts with tube rail barriers. A second portion of this option is to modify the sidewalk and bring it into code compliance by leveling and providing a suitable support system to prevent settlement. The City has the option of raising the sidewalk or lowering it. Raising the sidewalk without also raising Murray Blvd. would create parking issues with door opening clearances. Lowering the sidewalk provides for better car door access by re-establishing a typical 6 inch curb height along Murray Blvd. As a

“mountable curb” for vehicles that may stray from the roadway, this does have the potential to increase the lateral design load on the wall’s railing elements due to it being designed to resist today’s specified car impact loads.

In addition to the Boulevard side of the wall, renovation of the face of the wall and the vertical and horizontal joints would be recommended. There are certain areas that require standard concrete spall repair where significant sections of concrete have fractured and could compromise wall integrity during an entire face cleaning and repair. These areas should be replaced first, and then the entire face of the wall should be cleaned by high pressure water blasting followed by a re-facing the wall with a shotcrete system or a formed, cast-in-place new concrete face bonded to remaining sound concrete in the existing wall. The existing joints would then need to be re-established and filled with proper joint material, a hydrophilic sealer, or possibly pressure grouted to prevent continuation of fill washout from behind the wall.

The “foundation” renovation has multiple alternate solutions available to the City. All of them provide a similar result, in that they replace the weak link of the existing structural interface between the decaying timber piles, timber cribbing and the deteriorated concrete wall base that form the wall’s foundation. The timber pile head is severely deteriorated and will continue to degrade with prolonged exposure within the tidal zone. The same can be said for the concrete wall base that exhibited a loss of connectivity with the timber pile heads during examination. With the assistance of the geotechnical consultants, JMT has explored the options of micropiles, auger cast piles and steel HP’s as potential solutions for underpinning the concrete mass. These are further described in the following sections.

We have developed five conceptual design options and corresponding cost estimates for the seawall foundation rehabilitation. The existing Low Battery seawall stability and foundation improvements have been conceptually analyzed for expected loads including for flood loading based on current NOAA and FEMA design guides. These loads consider hydrostatic, hydrodynamic, breaking wave, and debris impact loading associated with the current design flood levels provided by JMT. The information used for the design is summarized in the following table:

Hydrostatic	Hydrodynamic	Breaking Wave	Debris Impact
1,897 plf @ 2.6' eccentricity	414 plf @ 2.8' eccentricity	7,897 plf @ 4.7' eccentricity	1,500 lbs @ 7.7' eccentricity

These load cases were combined using ASCE 7-10 suggested ASD load combinations. For the purpose of the conceptual design, the following load combinations were selected, as they will tend to govern the maximum tension, compression, and lateral loads on the foundation elements. However, for final design all AASHTO LRFD load combinations shall be analyzed for completeness and confirmation.

ASCE 7-10 Load Combination 2: $D + L + H$
ASCE 7-10 Load Combination 5: $D + 0.6H + 1.5F_a$
ASCE 7-10 Load Combination 7: $0.6D + 0.6H + 1.5F_a$

I. Micropile Installation

Option 1 (shown in Figure 1) utilizes pairs of vertical and battered micropiles to relieve the load on the existing timber piles. The battered piles would provide increased lateral load capacity to support the flood and saturated soil loading on the seawall. In the critical flood condition the battered micropiles would act in tension and the vertical micropiles would act in compression. The center bar would provide axial capacity to the micropiles and the bond zone would transfer the axial load into the stiff cooper marl. In one version of this option, the top of the micropiles would be integrated into the existing seawall and the load transferred to the piles through bond with the existing seawall once the core holes are grouted.

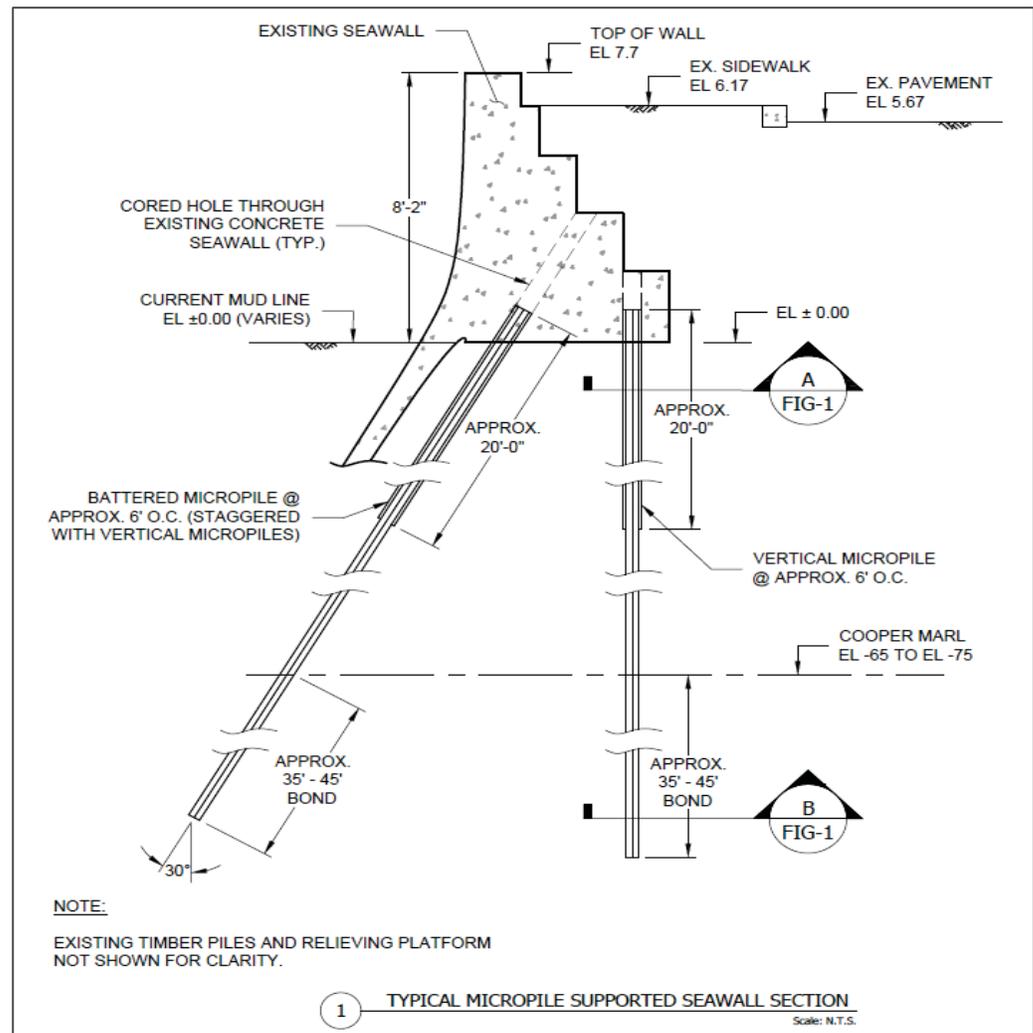


Figure 1

Option 1 assumes that the existing seawall concrete has adequate strength to transfer the loading into the piles. If further investigation indicates that the existing seawall concrete does not provide adequate strength to transfer the load, then Option 2 (shown in Figure 2) would be considered. The design of Option 2 is similar to Option 1 because it relies on the same layout of battered and vertical micropiles to support the lateral flood loads.

However, Option 2 would utilize a new concrete pile cap to transfer loading into the micropiles. This concrete pile cap would be bonded thru dowels into the existing seawall concrete and could reduce the amount of coring required during pile installation. Option 2 would also provide for additional settlement support of the sidewalk and reduced backfill thickness underneath the sidewalk.

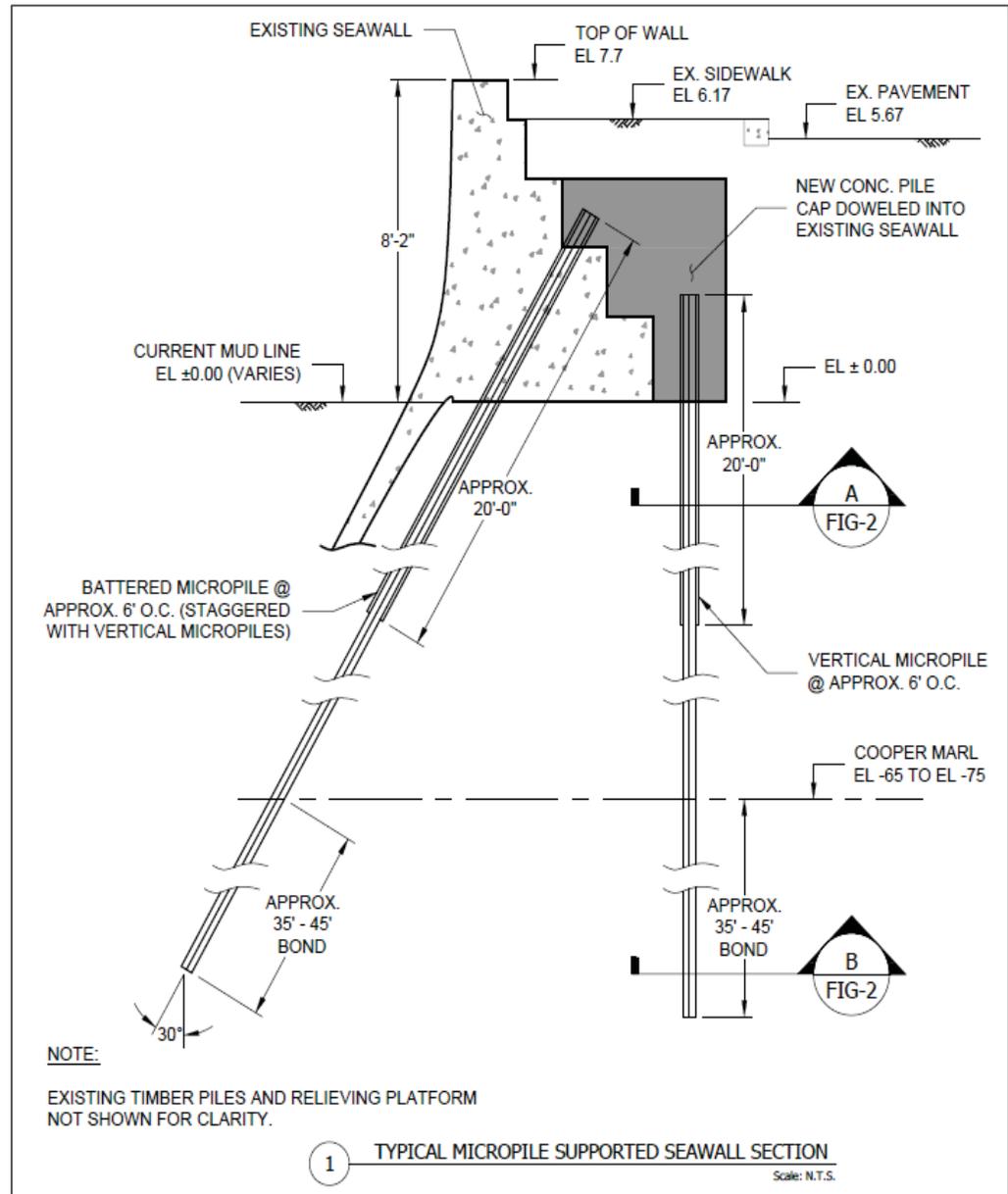


Figure 2

As an alternative to the battered micropile systems, Option 3 (shown in Figure 3) would utilize a pile-supported deadman to provide capacity to resist lateral loading. This option would transfer load from the seawall to the deadman through a pipe strut that would be installed below the road. The deadman would be constructed below the roadway median and would be supported laterally by passive earth pressure. Option 3 would eliminate the need for battered micropiles and would reduce the amount of required micropiles installed along the seawall. However, this option would increase the overall project limits of disturbance, limits of work, and required permanent easement below the roadway.

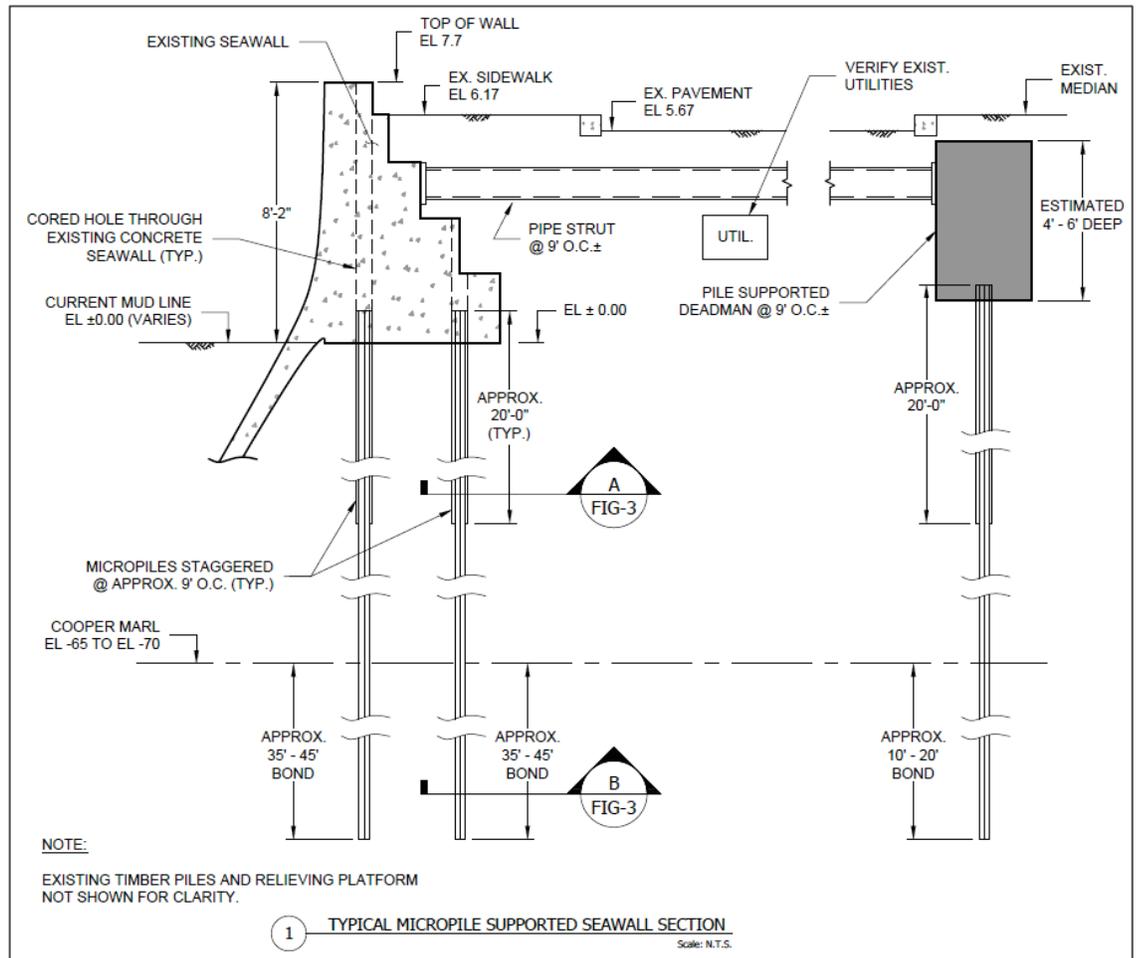


Figure 3

II. Auger cast Pile Installation

Option 4 (shown in Figure 4) is similar to Option 2, except that Option 4 would replace the vertical micropiles with auger cast piles. The use of continuous flight auger (CFA) piles in Option 4 could provide a simpler and less expensive alternative to the vertical micropiles specified in Options 1 and 2. However, mobilization of two different pile systems and the space requirements for installation of the CFA piles may have additional impacts to the project.

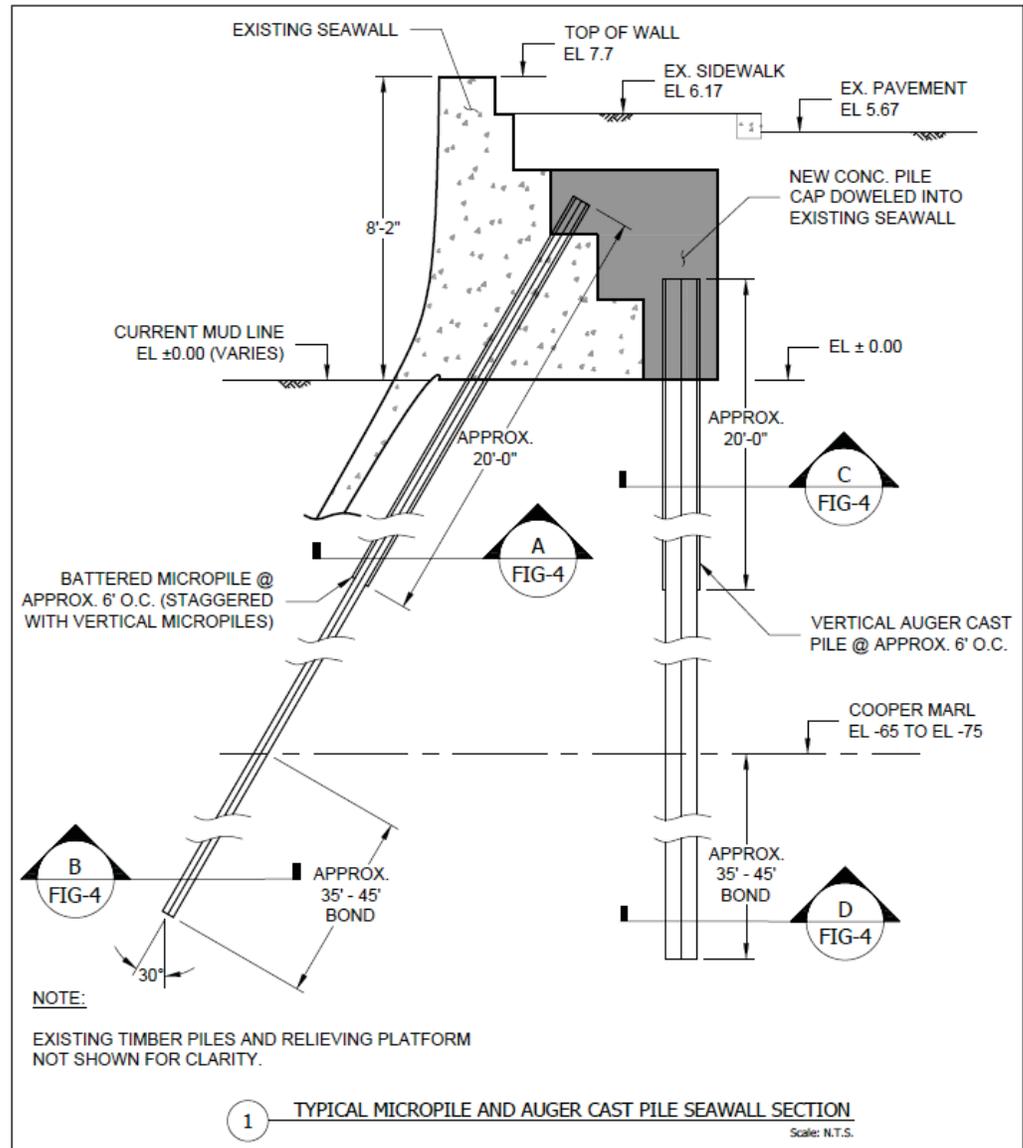


Figure 4

III. Steel H-Pile Installation

Option 5 (shown in Figure 5) is similar to Option 2, except that Option 5 would replace the vertical micropiles with steel "H" piles. The use of a conventional steel H-pile installed with a vibratory hammer could provide a simpler and less costly alternative to the vertical micropiles specified in Options 1 and 2. However, mobilization of two different pile systems and the space requirements for installation equipment necessary for the HP pile installation may have added impacts to the project. The driven pile option would likely have the largest impact on the nearby residences through vibration and noise vs. the more surgical drilled in micropiles or auger-cast piles.

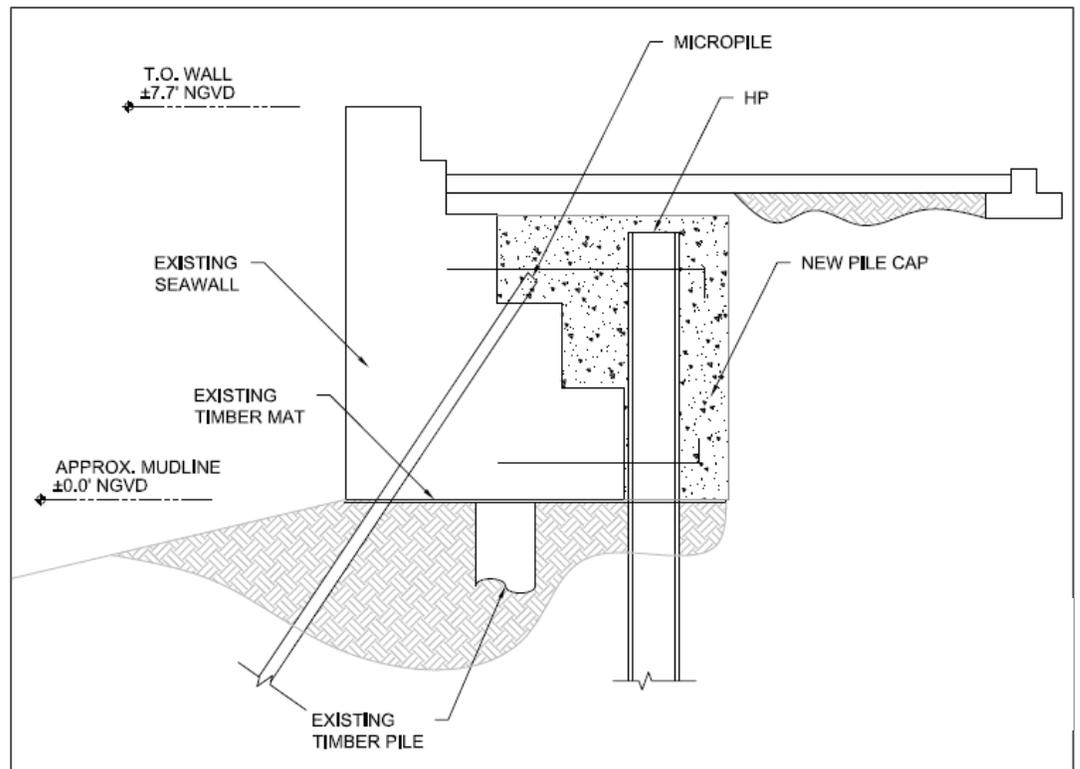


Figure 5