3.0 Condition Assessment

3.1 Site investigation

a) Survey & Site Geometry

GEL Engineering, LLC (GEL) performed surveys in support of the Low Battery investigation project. The surveys included upland and marine data collection that were integrated to create a base map of the project area. The surveys utilized existing nearby National Geodetic Survey (NGS) monuments in conjunction with Global Positioning Systems (GPS) to establish horizontal and vertical control for the project.

Subsequent to the survey control procedure, a survey of the project area was performed to identify the location of existing infrastructure, above and below ground utilities, rights of way, and the Ocean and Coastal Resource Management (OCRM) critical line. Survey points were collected in X, Y, Z 3D format utilizing Trimble Robotic Total Stations and Trimble R8 RTK GPS receivers. Sub-surface utility locates were performed by GEL Geophysics, LLC and mapped by GEL surveyors to tie in all the survey aspects of the project. Invert elevations of storm drains and sewers were collected, when available, during early morning hours (4-5 am) to avoid conflicts with traffic and tourists along the battery.

GEL also performed the hydrographic survey of the bathymetric conditions in accordance with the United States Army Corps of Engineers (USACE) Hydrographic Surveying Manual, EM1110-2-1003. The hydrographic survey project area included approximately 4,800 LF along the length of the Low Battery and 50' seaward of the exterior battery wall. All hydrographic surveys and bathymetric mapping utilized Real Time Kinematics (RTK) GPS for navigation and data collection. Sounding data was collected using a Teledyne Odom CVM mobile single-beam echo-sounder operating at 200 kHz with a 6.0 degree transducer mounted over the side of the survey vessel. Positioning and soundings were processed with Hypack Hydrographic Surveying Software in computers mounted in the survey vessel. The data from each survey was integrated into a single data base and a single AutoCAD Civil 3D file.

<u>High Definition 3D Scanning</u>

JMT performed a high definition 3D Static Scan of 4,800 linear feet of the Low Battery, Murray Blvd., and the existing homes and businesses that front Murray Blvd using a Leica Scan Station C10. JMT completed 67 individual static scan world setups, and each scan provided a 360 degree horizontal scan with a 90° vertical scan. The scanned data was referenced horizontally to survey controls established and provided by GEL.

The obtained 3D Point Cloud data was post-processed & unified using Leica Cyclone 8.0. A LAS file was exported and brought into TopoDot where all line work and spot grades were created. From this it was brought into Carlson Survey/AutoCad and 3D surface was created to include all striping, curb & gutter, the wall face, edge of pavement, existing dwellings along the corridor.

Items of note from the survey data collected are as follows:

1. Seawater elevations are -2.16' MLLW, -1.97' MLW 3.25' MHW and 3.60' MHHW (NGVD29)

2. The top of the seawall (along the railing line) ranged from 7.63' to 7.96'. The typical average was 7.7' (NGVD29) leaving approximately 4.1 ft. of freeboard during a typical high tide.
3. The sidewalk elevation varies in elevation from 5.48' to 7.01' (NGVD29) along the interface with the wall and from 4.30' to 6.62' (NGVD29) along the curb line.
4. Murray Boulevard varies in elevation from 4.18' to 6.75' (NGVD29) along the curb line nearest the battery wall and from 3.96' to 6.74' (NGVD29) along the median curb.
5. The mudline along the face of the seawall varies in elevation from -2.66' to 2.78' (NGVD29)
6. The exposed wall height varies from 4.05' to 9.17' above ground.

b) Accessibility/ADA

A significant length of the sidewalk between Station 1+00 and Station 13+00 has settled over the years. The settlement was most visible along the curb line of Murray Blvd. while the seawall supported the sidewalk edge adjacent to the railing. This created differential settlement of the sidewalk which can be seen in the following photographs taken during the site investigation on June 18, 2015.

Approximate Station 1+00 looking up station

Approximate Station 13+00 looking down station

This differential settlement as well as other geometrical issues raised concerns about the existing sidewalk not meeting Americans with Disabilities Act (ADA) requirements for a public access walkway. ADA requirements for public access walkways are summarized below as compared to the currently documented condition of the Low Battery sidewalk.

Current ADA standard	Current Geometry of Battery sidewalk
maximum cross slope of 2%	maximum noted cross slope of 29%
maximum running slope at 5%	maximum running slope at 1.75%
minimum clear width of 36”	minimum width of ~10’
maximum vertical level change of 1/4”	maximum vertical level change 1/4”.

In addition to the above mentioned geometrical issues, it was noted that there were only two ADA curb ramps / access points for crossing Murray Blvd. Each of these locations (one at the intersection of King and the other at the midpoint of Whitepoint Gardens) was not in compliance with the current ADA standards. The ramp slopes exceeded the allowable 1:12 (8.33%) ratio and did not have level (2% or less) landing areas at the top of the ramps.

c) Utilities

While on site for the topographical surveying, GEL also surveyed all of the underground and above ground utilities within the project area. The survey identified mostly subsurface utilities including an existing waterline, storm sewer line, and power line that ran underneath the Murray Boulevard median. The only above ground utility within the project site was the light posts located in the median.

The north side or landward side of Murray Boulevard contained both a sanitary sewer and a natural gas line. Most of the manhole covers were seized closed and could not be opened. A few manholes were opened and the sanitary sewer consisted of an 8" clay pipe. Around Station 21+50, the sanitary pipe has been abandoned in place due to being crushed. It has since been replaced by another 8" clay pipe. The natural gas line was only present from Station 8+00 to 23+00.

A water force main ran just to the south of the storm sewer line along the Murray Boulevard median from approximately Station 0+00 to 39+50 where it took a 90º turn to the north, up King Street. At several intervals the water main crossed the right of way to service the residential homes located on the north side of the road.

There were no utilities identified along the seawall side of Murray Boulevard, and none that cross the center median. There should be no utility interference during construction of any seawall improvements as long as the work area is contained to the south side of the median.

d) Drainage Accommodations

All of the surface drainage for the site were collected in curb inlets that ran along both the north and south curb lines of Murray Blvd. and conveyed to the collection line that ran under the median. Beginning at Station 2+58, curb inlets were found along the seawall side curb line at approximately 165' offsets. The curb inlets connected back to the main storm water sewer system with a 10" clay pipes, although some have been replaced in recent years with a 15" reinforced concrete pipe. Several of these pipes were completely full of dirt and could not be completely evaluated. Curb inlets were also present on the north side of Murray Boulevard at 165' offsets connected back to the main storm water sewer with 10" clay pipes. The main storm water sewer consisted of an 18" clay pipe running directly under the vegetated median in the center for the length of the 70ft right-of-way of Murray Boulevard until approximately Station 40+00, where it diverged to the south side of the right-of-way.

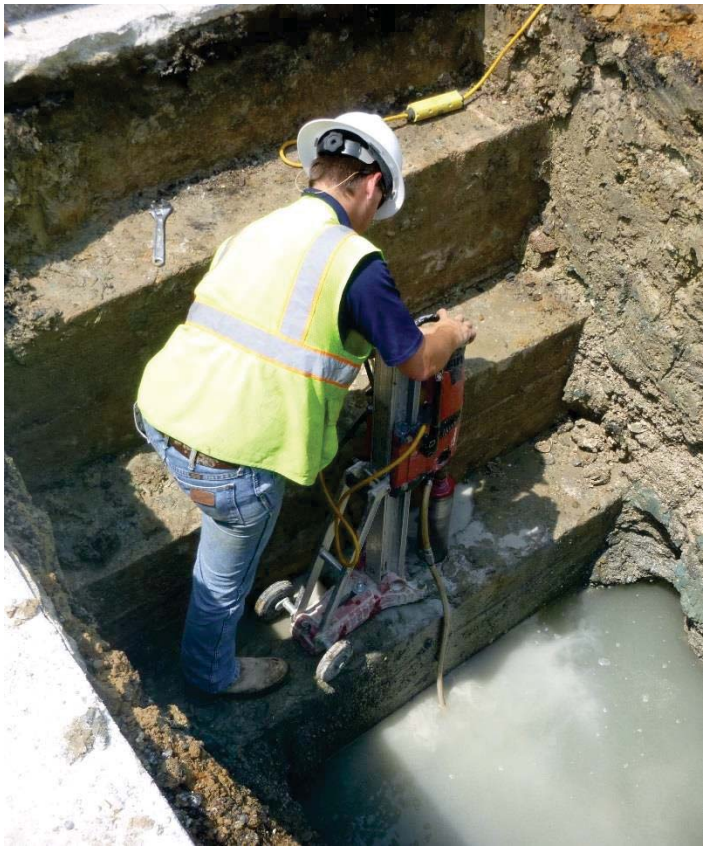
The storm water collector line, then distributed the water through seven wall penetrations, or weep holes, at various places along the seawall. During construction, the drainage penetrations of the wall will have to be maintained and may need to be repaired or upgraded as necessary.

3.2 Material testing

a) Concrete Cores

Four strategically placed test pits were excavated on the landward side of the seawall. The location of the pits were chosen to be representative of the various phases of wall construction over the years. This provided confirmation of the size and shape of the seawall depicted on the original drawings obtained from the archives. The test pits remained within the limits of the sidewalk to avoid disruption of traffic and exposed the buried face of the seawall for inspection. Exposing the existing structure also

provided access to collect concrete cores and samples from the timber substructure for further testing. Five concrete cores in total were collected at the project site, four from the test pits and one from a designated location along the seaward face of the wall.



Concrete core sample being taken from landward side of wall after excavation

Under the supervision of Terracon, City staff removed the concrete sidewalk and excavated the fill material to the required depth where Terracon collected cores and sample. The concrete samples underwent a series of laboratory tests to determine in-place properties for engineering evaluation of the structure and for determining competence of the existing materials by a specialty testing company. All test results can be found in Appendix B of this report.

- Petrographic analysis of Concrete (ASTM C856/C457)
- Compressive strength testing (ASTM C39)
- Density of hardened concrete (ASTM C642)

Concrete core sample submitted to laboratory for analysis

The concrete samples showed the wall concrete to be in fair condition. Deterioration was observed on the top surface of the cores taken from test pits and on the outer surface of the core taken from the outside of the wall. Petrographic analysis indicated that this deterioration was due to sulfate attack caused by extended exposure to seawater, with secondary sulfate deposits characteristic of marine-sulfate attack of the concrete. The wall cores were further compromised by severe corrosion of the adjacent reinforcing steel. In general, the coarse and fine aggregates were found to be hard, sound and durable, where the residual cement particles were relatively coarse in comparison to modern cement samples.

The concrete cores were also tested for compressive strength and density utilizing ASTM C39 and C642, Standard Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete. Three of the cores, two from the wall and one from the wall face, came back with strengths ranging from 4,120 psi to 7,220 psi with corresponding densities over 140 pcf. Two of the samples (from Station 40+12 and Station 04+79) had strengths in the 2,840 to 3,150 psi range with corresponding densities below 140 pcf. The petrographic tests reveal that the core samples from 40+12 and 04+79 evidence of incomplete mixing. Large balls of neat cement were observed, indicating the cement was not evenly distributed. This is the likely culprit of the lower strength breaks and the lower densities. During the era that the wall was built, quality control and quality assurance measures, along the concrete mixing techniques, were not always consistent from one batch to the next. In accordance

with American Concrete Institute (ACI) 301, the strength results of the concrete tests permit the existing wall to be analyzed as 3000psi concrete.

b) Timber Sampling & Testing

At the test pit locations, wood samples were taken from the timber piles and from the timber cribbing used as a base form for the concrete wall construction. As the test pits were excavated, a Schnabel representative was on site to extract the samples and forward them to Wood Advisory Services (WAS) to be evaluated for marine borer activity and microbiological deterioration. The marine borers, both Teredo and Limnoria, leave obvious bore holes, visible to the eye. While microbiological decay such as, brown, white and soft rot were detected by examination under a microscope. These rots breakdown the cellular structure of the wood reducing its strength.

Based on the evaluation performed by WAS, the tested timber piles showed signs of very significant soft rot fungi causing heavy cell structure deterioration. This level of cellular structure decay has left the wood cells unrecognizable and does not provide structural support. The timber platform samples that were tested displayed a combination of moderate to heavy brown and soft rot fungi and poor cell structure, a condition where most of the wood cells have been compromised. These findings coincide with reduced strength properties in both the timber piles and the platform. The full report from WAS can be found in Appendix C of this report.

These results confirmed the suspicions of the City, and from what was witnessed during the construction of “the turn”, which the timber foundations were in very poor condition and not providing the structural support to the wall it was as originally intended. The deterioration of the pile heads at the interface with the wall indicates that the lateral capacity of the wall is compromised and that the piles are mainly carrying vertical gravity loads. Additionally, the deterioration of the pile-footing interface has greatly reduced the vertical load capacity of the piles, leaving them susceptible to failure if they were to be subjected to a sustained, or increased loading situation.

c) Geotechnical Investigation

Terracon provided material testing and sampling services as part of the geotechnical investigation. As discussed earlier, the complete investigation report can be found in Appendix B. The soils investigation utilized ten (10) Cone Penetration Tests (CPT) and two (2) Soil Test Borings (STB). All testing locations were coordinated with the City by JMT and located in the field by both JMT and Terracon by taking measurements from existing survey markings. A field log of each STB and CPT were prepared by field personnel and included visual classifications of the materials encountered during drilling as well as the driller’s interpretation of the subsurface conditions between samples.

Borings B-11 and B-12 were advanced to depths of 100 feet and 75 feet, respectively, below the ground surface using rotary wash drilling techniques. Soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., splitbarrel sampler, also known as standard split-spoon. The sampler was advanced into the soil a total of 18 inches by striking the drill rod using a 140-pound safety or automatic hammer falling 30 inches. The greater portion of both test borings were found to have a large amount of sandy silt which provides very little lateral resistance for piles and foundations.
The CPT soundings provide a multitude of undisturbed field data, including tip resistance, sleeve friction, and pore pressure. Based on the report from Terracon, it can be inferred that the optimal

bearing layer is approximately 60' to 70' below the mudline. Several of the CPT soundings uncovered a large amount of organic and silty clays, as well as soft silty sands. Again, these types of materials provide very little lateral resistance for piles and foundations unless the piles are carefully seated on the bearing layer located in the marl strata.

Terracon also performed soil testing of materials sampled from the site, where the results are contained in their report found in Appendix B of this report. The soils were tested for possible increased corrosive effects, which included:

- Chloride-water soluble testing (AASHTO T-291/ASTM D1140)
- pH Testing (AASHTO T298-91)
- Resistivity Testing (AASHTO T288-91)
- Sulfate-Water Soluble Testing (AASHTO T290-91/ASTM D4327)

The corrosivity testing revealed the soils presented a heightened risk of corrosion for exposed metals. This increase is typical of clay soils in the Charleston area. Any exposed metals would require a protective covering, such as coal tar epoxy coating, concrete encasement or alternately, additional sacrificial material thickness of the structural member. Galvanizing is not recommended for corrosive soils applications because the inclusion of additional metals often found in soils will consume the zinc coating at an accelerated rate.

Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivity result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is (Romanoff, 1989):

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage. Based on local experience and current practice the most cost effective protection choice using a steel H pile underpinning would be the combination of a thicker cross section pile and a protective epoxy coating. Other underpinning options are concrete encased, which would provide the necessary protection.

Below the water table the corrosion of the steel is minimal and is not considered to be significant, above the water line and embedded in fill soils, the corrosion rate is variable and usually not considered serious. The free oxygen exposure and inclusion of salt water is the worst case scenario for steel piling. Current best practices recommend the use coal tar epoxy coating, and/or additional sacrificial thickness to the steel element.

From the historic documents available in the City archives, a test pile record was acquired. The pile driving record describes pertinent information about the installation of the original foundations. The contractor used a 3000lb drop hammer to install timber piles with an overall length of 51 feet to 61 feet. The number of hammer blows per foot were recorded along with the total depth of pile penetration below the mudline. The synopsis follows:

- Station 1 depth driven 37'-5¾"
- Station 22 depth driven 35'-0"
- Station 27 depth driven 40'-5¾"
- Station 32 depth driven 35'-0¾"
- Station 35-36 depth driven 47'-0¾"

Based on the geotechnical testing, it has been determined that the existing foundation piling were not driven into a bearing strata (such as the Cooper marl) capable of supporting the wall during a seismic event.

While the wall's foundations have performed well to date, the extensive deterioration of the timber piles and the findings of their not being embedded into the marl, require consideration for replacing the foundations in-place if possible, or possibly reconstructing the wall.

3.3 Visual Structural Inspection

a) Overall Condition & Inspection Findings

JMT performed onsite investigations to evaluate the existing condition of the seawall. Two engineers licensed in the state of South Carolina visually inspected the entire length of the wall which included the wall seaward face, parapet, railings and sidewalk. No destructive testing was performed during this operation, although this investigation located areas of interest which led to further testing and investigation by Schnabel and Terracon. All substantial defects were photo-documented and these photographs were named according to the stationing of the wall where they were located. A description of the deficiencies uncovered was recorded and the list of deficiencies, along with their locations, are documented in Appendix E. This information could prove useful for determining locations that may need more in depth investigation and for future comparative inspections of the wall.

Typically, along the entire length of the wall, the surface concrete has deteriorated at, and below, the mean high waterline. This deterioration is an effect of the aggressive saltwater environment and years of tidal waters eroding the cement/sand paste of the concrete combined with the age of the structure and material properties used during construction.

Station 47+65 looking up station toward "The Turn". This shows the deterioration of the concrete. The cement/sand component is missing, leaving the large aggregate protruding.

Station 40+73 looking down station toward the Coast Guard base. Again showing the deterioration of the cement/sand paste leaving large aggregate.

Previous repairs to the concrete, specifically the upper curb, were poorly executed. The replaced concrete was typically installed with a feathered edge, rather than square cutting the existing concrete. This feathered edge generally does not have the proper mix of components and usually cracks or spalls in a relatively short amount of time. The concrete was poorly consolidated during placement as displayed by numerous areas with large voids known as "honeycombing". This could be exacerbating the weakness of the feathered edge.

Station 4+36 Shows poor repair technique, the new concrete is not adhering well to the existing concrete and the formwork anchors were left behind.

In a number of locations along the water side face of the wall, bolts, bolt holes, and embedded threading devices were left in place from previous repair projects. These were all corroded and deteriorated. The corrosion of the bolts causes swelling of the embedded portion of the anchor and eventually spalls the concrete off of the face. The accepted procedure for removal of these anchors is to cut them off with a torch and burn the top few inches of the bolt down below the surface. The remaining hole should be filled with a cementitous repair epoxy. The epoxy bonds to the existing concrete and permanently seals the embedded corrodible iron products, and prevents corrosion due to salt water and atmospheric exposure. This prevents future spalling damage from corrosion or biological growth and even freeze/ thaw damage.

Bolt holes for formwork of a previous repair area.

Station 32+02. Note the rust stains leaching from a site of an embedment anchor that was not properly patched after the removal of the bolt.

Poor concrete work on previous repair

A number of the granite rail posts were experiencing deterioration at the connection point to the wall. Based on research of historical documents and on site investigation, the rail posts are attached by embedding the stone post into a notch in the wall and then packing the area around the post with a pea-gravel concrete or grout. In many locations there is severe deterioration of the concrete at the embedment and the railing post is no longer anchored securely. This creates an unsafe situation in a tourist populated area where people are very likely to lean on and/ or look over the rail.

Station 47+06 an example of the deteriorated embedded granite barrier post.

The concrete rail posts that are used for the majority of the Low Battery barrier are also in a degraded state. The reinforcing bars that were used to affix the post to the curb have corroded and caused spalls to varying degree in many of the posts. A number of the posts have been repaired and replaced in the past.

Station 27+95, an example of the deteriorated concrete rail post.

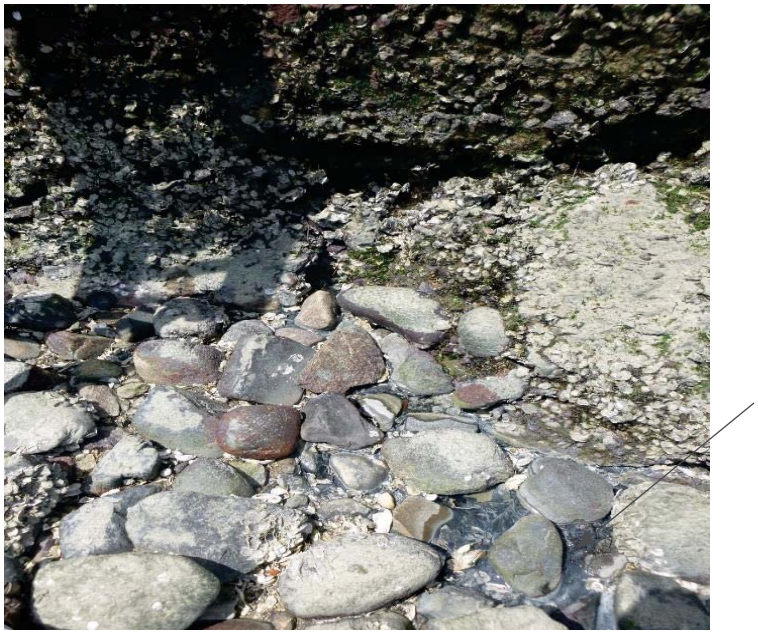
Throughout the length of the 4,800 feet of wall the top curb was deteriorated, cracked, and leaching rust stains. As discussed prior, the repair methods used by others was likely not performed according to standard American Concrete Institute (ACI) procedures. The curb is one of the few places where steel reinforcing was used in the existing construction. The curb is also in the location of the wall that receives the majority of salt spray from breaking waves in the harbor. This is considered to be the most corrosive zone for ferric items in an aggressive salt water environment. It is our opinion that the curb should be removed in its entirety and replaced.

Station 43+42 Spalled and cracked curb.

Station 26+23 Cracked curb. This section is actually a repair, most likely from sometime in the 1970's. Note the rounded edge aggregate that is now exposed.

Station 48+00 Top of curb and concrete post spalling.

A major source of concern was evidence of water entering the joints of the wall and weeping soil material through the opened crack. Generally there was a single horizontal joint across the entire wall surface and vertical joints spaced at various distances along the wall. Some of the vertical joints have experienced considerable damage, the cause could be contributed to numerous issues. Thermal expansion and contraction are the most likely the culprit, along with organism growth, scour and chemical attack.



Station 40+12: Notice gray soil material weeping from joint in wall.

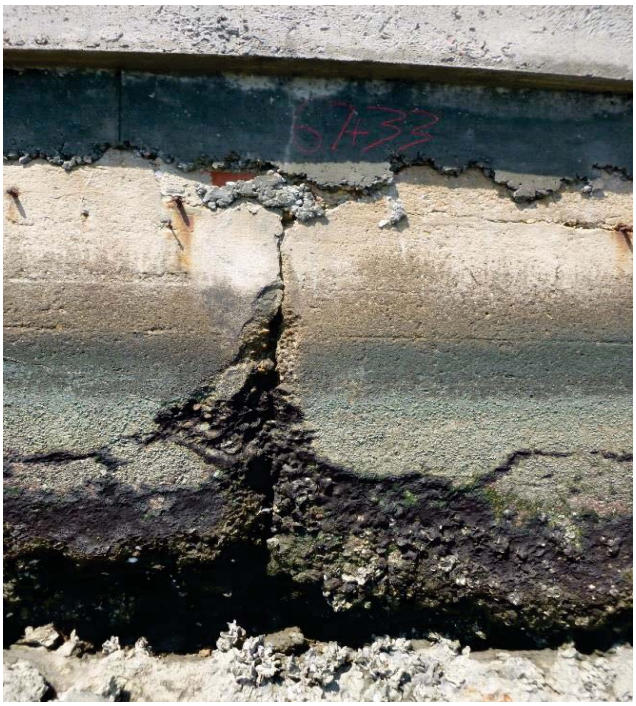
Station 7+22 an example of the spalling that is occurring in the horizontal and vertical joints.

During the field investigation, in a few areas, diagonal cracking in the wall was visible. This suggests the wall has undergone differential settlement, most likely due to failure of the pile heads in certain locations nearby. Based on engineering principles, it is assumed that as the pile head weakens, it is being crushed by the load of the wall and a section of the wall sinks lower, causing the wall to bend. This induces a tension force in the wall and, because the wall is unreinforced and concrete is weak in tension, a diagonal crack appears.

Station 3+27 Diagonal cracking in the wall.

Station 9+12 Diagonal crack and undercutting at the base of the wall.

There was a substantial amount of undercutting at the base of the wall as shown in the photographs below. It is likely that these areas are also a source of water entering behind the wall and causing soil wash out from behind the wall. The cause for the deterioration is unknown at this time. It is most likely due to chemical attack.



Station 4+53 Note the deep undercutting of the wall.

Station 9+12 Note the deep undercutting of the wall.

There were also concerns about the existing sidewalk, in certain areas where it did not meet ADA requirements for a public access walkway. Americans with Disabilities Act sets the maximum cross slope at 2%. In the region between Station 1+00 and Station 13+00 the cross slope measures approximately 3-1/2:12 or 29%. In addition, there were also areas that were possible trip hazards. Many other locations of the seawall where the sidewalk has been modified by grinding, to create a smooth surface for walking.

Approximate Station 1+00 looking up station

Approximate Station 13+00 looking down station

3.4 Lifecycle Discussion

Most marine structures, due to the increased cost of initial construction and maintenance, are designed for a longer life expectancy. SCDOT standard design life for a cast in place concrete bridge is 75 years. Given that the existing wall has been in use for more than 100 years suggests the original design concept of the wall is sound. However with modern materials and construction knowledge it is possible to extend the life of the core wall by reshoring and rebuilding the deteriorated components.

Any newly placed concrete would be specified to contain corrosion inhibiting admixture. Corrosion inhibiting admixtures are liquid chemicals that increase the passivation of the ferric reinforcement in the concrete. The admixtures are added during the concrete batching process and prevents the attraction of the chloride ions to the iron content of the steel reinforcing. This can significantly reduce the maintenance costs of reinforced concrete throughout a typical service life. Although the inhibitor can raise the corrosion threshold they are not an alternative to a properly placed, durable concrete.

Another admixture included in the concrete would be silica fume. This is a powder like substance that has a particulate size much smaller than the cement powder in the concrete. The silica fume acts to fill any voids in the concrete making the in place concrete more dense and less permeable.

For the reinforcing option available, plain deformed bars, galvanized, epoxy coated, stainless steel, and Fiber Reinforced Polymers (GFRP) are available as options. Each option has positive and negative attributes. The installed cost/benefit should also be considered for future design. Plain reinforcing steel will have the lowest cost, but the shortest anticipated design life. Stainless steel has been historically proven to be the longest lasting of the steel options but that longer life comes with an extremely high cost. The FRP reinforcing is a relatively new option available, the historical in place data is not available, however the theory is sound and has become increasingly popular in cold regions

for bridge decks subject to corrosive deicing applications. The FRP does not have any corrodible material, and when encased in concrete it is protected from UV decay, it could have the longest life at a cast similar to galvanized.

It is conceivable that the target design life for city options should be 100 years. The complete demolition and replacement option would allow for a reasonable expectancy of another 100 years. Keeping in mind that design life does not mean maintenance free life. Completely rebuilding the wall would provide modern materials and modern quality control and construction techniques that are more readily accepted for predicting service life.

In the event that the under pinning and renovation option is chosen, it is important to know that the extended service life would be more difficult to estimate. The portions that are rebuilt, the foundation and the curb and rail, could be approximated. Issues arising from existing sections of wall are more difficult to assess and predict. The renovation plan essentially leaves the existing concrete mass and restores the peripheral components. The existing concrete mass appeared to be stable during the field investigation, however that assessment is limited to just the areas inspected.