SEAWALL EVALUATION AND STUDY

for the

City of Charleston, South Carolina

PHASE I REPORT CONDITION ASSESSMENT

PHASE II REPORT HISTORIC RESEARCH

PHASE III REPORT DETAILED CONDITION INVESTIGATION

PHASE IV REPORT COMPREHENSIVE ENGINEERING ANALYSIS AND REPAIR RECOMMENDATION

September 22, 2004

Cummings & McCrady, Inc.
Charleston, SC

The Sheridan Corporation
Charleston, SC

4SE, Inc.
Charleston, SC

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AND REPAIR RECOMMENDATIONS

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Executive Summary

On August 7, 2002, the City of Charleston, South Carolina retained Cummings & McCrady, Inc. of Charleston, South Carolina as the general consultant for the study and repair of the city's various historic seawalls. Cummings & McCrady, Inc. subsequently retained local consulting engineering firms to provide specialized expertise including The Sheridan Corporation for structural engineering of waterfront structures, 4SE, Inc. for structural engineering of historic structures, Soil Consultants, Inc. for geotechnical engineering, and Forsberg Engineering and Surveying, Inc. for topographic surveying.

The historic seawalls are a defining feature of Charleston. The seawalls are showing definite signs of deterioration from long-term exposure to the elements. The first step in the seawall repair project is to conduct a comprehensive study of the existing conditions of the seawalls. The primary purpose of the study is to establish the requirements necessary for the seawalls to be repaired. The secondary purpose is to establish baseline conditions for use in any future monitoring of the seawalls' structural conditions.

The historic seawalls included in this study are the High Battery, the Low Battery, the seawalls surrounding Colonial Lake, and the portion of The City Marina seawall immediately adjacent to Lockwood Boulevard.

The seawall study has the following objectives:

1. To document and assess the general existing conditions of the seawalls;
2. To provide conceptual repair and/or stabilization options and recommendations;
3. To prepare estimated costs of construction; and
4. To set priorities for the repairs.

Thus, the seawall study becomes the basis on which subsequent cost/benefit analyses can be performed, and if determined to be a viable project, the starting point for the far more comprehensive design phase.

The seawall study is subdivided into the following phases:

Phase I is the general condition assessment phase.
Phase II is the historic research phase.
Phase III is the detailed investigation phase.
Phase IV is the comprehensive engineering analysis and repair recommendation phase.

The scope of services stipulates that the seawalls of the High Battery, the Low Battery, and the Marina seawall immediately adjacent to Lockwood Drive shall be included in all of the four phases of the study. The seawalls surrounding Colonial Lake shall be included in the first two phases of study only.

After each phase of the study is completed, an interim report will be submitted based on the results of that phase of the study. When the entire study is complete, a comprehensive final report will be issued.

Executive Summary Phase I – The General Condition Assessment Phase

The categories used in this report for assessing the structural condition of the waterfront structures are "Good", "Fair", and "Poor". For concrete, these overall evaluation categories are based on the guidelines outlined in the reference manual Protection, Inspection, and Maintenance of Marine Structures by Pile Buck, Inc., 1990, and the Bridge Inspector's Manual/90 by the Federal Highway Administration, July 1991. For stone masonry, the evaluation will parallel the concrete criteria.

In general, "Good" condition means that the structure only has minor defects; "Fair" condition means the structure has moderate defects; and "Poor" condition means that the structure has major or severe defects. When it is difficult to put the structure completely into one category, a term such as "Fair to Poor" condition is used to signify such a marginal call. It should be recognized that in borderline cases, two different engineers could easily rate the same structure in different categories. The evaluation of the point at which a "minor" defect becomes a "moderate" defect, or the point at which enough "moderate" defects become a "major" defect is subjective. Even a condition assessment of a measurable section loss is subjective based on what is considered a "permissible" section loss. In this report, we feel the various overall condition assessments of the structures have been made with a conservative viewpoint.

For the longer seawalls, if conditions vary significantly within their length, the condition assessment will be zoned accordingly with reference to a specific street address, the nearest cross street, or other prominent landmark.

It needs to be emphasized that the condition assessment is only preliminary. The condition assessment is not an evaluation of structural capacity. The determination of such would necessitate a structural analysis.

On November 7, November 15, and November 22, 2002, Mr. John Sheridan IV, P.E. from the Sheridan Corporation, and Mr. Craig Bennett Jr., P.E. from 4SE, Inc. performed the "walk-by" and "float-by" visual inspection of the specified seawalls. Mr. Jerome English, AIA, project manager from Cummings & McCrady, Inc. was intermittently present during various stages of the inspection.

OVERALL GENERAL CONDITION ASSESSMENT OF THE HIGH BATTERY SEAWALL

1. The flagstone walkway (includes the flagstones, the mortar joints between adjacent stones, and the "supporting" underlying fill) = POOR.
2. The individual flagstones = GOOD to FAIR.
3. The stone masonry portion of the seaward seawall (includes the stones and the mortar joints between adjacent stones) = FAIR.
4. The individual stones composing the seaward seawall = GOOD.
5. At the south end, the concrete portion of the seaward seawall = POOR.
6. At the south end, the concrete portion of the walkway = POOR.
7. The landside seawall = FAIR.
8. The railing (does not meet current safety standards) = GOOD to FAIR.

OVERALL GENERAL CONDITION ASSESSMENT OF THE LOW BATTERY SEAWALL

1. The concrete sidewalk (from the eastern end at White Point Gardens to King Street) = GOOD.
2. The concrete sidewalk (from King Street to Tradd Street and includes the concrete slabs and the settlement of the underlying subgrade) = POOR.
3. The concrete coping (from the eastern end at White Point Gardens to King Street) = POOR.
4. The concrete coping (from King Street to Tradd Street) = VARIES between GOOD and POOR.
5. The railing (does not meet current safety standards) = FAIR.
6. The exposed seaward face of the seawall (from the eastern end at White Point Gardens to King Street) = POOR.
7. The exposed seaward face of the seawall (from King Street to Tradd Street) = FAIR to POOR.

OVERALL GENERAL CONDITION ASSESSMENT OF THE MARINA SEAWALL

1. The marina seawall = POOR.
2. The landside of the marina seawall = POOR.

OVERALL GENERAL CONDITION ASSESSMENT OF THE SEAWALLS SURROUNDING COLONIAL LAKE

1. The seawalls surrounding Colonial Lake = POOR to FAIR.
2. The landside of the seawalls surrounding Colonial Lake = GOOD.

The purpose of the general condition assessment phase of the seawall study was to detect any obvious major damage or deterioration and to provide initial data for the subsequent detailed investigation phase. Given the length of time these historic seawalls have been exposed to the harsh marine environment, the nature and degree of the deterioration observed was generally consistent

with what was anticipated.

With Phase I of the seawall study complete, the observed physical conditions of the historic seawalls are better known and provide the starting point for the subsequent phases of the study.

The next phase of the study is the historic research into the original engineering design and construction of the historic seawalls. The purpose is to provide as much structural background information about the seawalls as possible in order to supplement and fine-tune the planning for the subsequent detailed investigation phase.

Remaining are many questions to be answered before any meaningful conclusions can be made regarding repairs to the historic seawalls.

Submitted is a full report on the general condition assessment phase of the seawall study for the City of Charleston, South Carolina.

Executive Summary Phase II – The Historic Research Phase

The City Of Charleston Archives, The Charleston Library Society, and the South Carolina Room of the Charleston County Library were the archival sources of historic information utilized herein.

In 1804, a severe storm swept away the palmetto log seawall that extended along most of the frontage of the present day High Battery seawall. Its replacement was a seawall built with stone ballast from arriving ships. Eventually, the stone seawall was completed circa 1820. In 1893, a major hurricane, with a reported thirteen foot tidal surge, impacted the Charleston peninsula. Great consideration was given to strengthening and rebuilding the High Battery seawall to better withstand major hurricanes. The 1893 plan called for providing interior concrete walls to be located under the walkway to reinforce both the stone masonry seaward seawall and the foundation wall of the landside seawall.

The basic structure of the stone masonry portion of the High Battery seawall appears to be essentially unchanged since its last major reconstruction in 1893. Nevertheless, 110 years of exposure to the harsh marine environment is a long time and, at present, the seawall shows signs of deterioration.

The present Low Battery seawall was constructed in two distinct phases as part of the overall “West End Improvement”, a large land reclamation project. “The Boulevard” was the first phase of the “West End Improvement” and was constructed between 1909 and 1911. During this phase, the seawall was constructed from the foot of King Street to the west end of Tradd Street. The second phase, known as “The Boulevard Extension”, was constructed between 1917 and 1919. The seawall was constructed between the foot of King Street to just past the southeastern tip of White Point Garden. Near its eastern end, the seawall was increased in height and curved northward forming the

concrete extension at the south end of the High Battery seawall.

The entire seaward face of the Low Battery seawall and the concrete extension at the south end of the High battery seawall is skirted with an array of timber sheet piles forming part of the retaining wall system to retain the landside fill. The existing condition of the timber sheet piles, the timber platform deck, the timber support beams, the steel bolts, and the top portion of the timber support piles are very much a concern.

It is believed that the Marina seawall, with its proximity to the West Point Rice Mill, was constructed under private ownership as a wharf for the West Point Rice Mill facility. It is also believed that this seawall was completed in the time period between 1844 and 1855.

In 1882, a concrete wall was constructed along three sides of Colonial Lake adjacent to Rutledge Avenue, Broad Street, and Beaufain Street. Because necessary additional appropriations were not immediately forthcoming, the complete enclosure of the lake with concrete walls was not finished until 1884.

The Marina seawall and the seawalls surrounding Colonial Lake have not been renovated since their original construction and display significant signs of deterioration.

With Phase II of the seawall study complete, the original engineering designs and/or the historic period of construction of seawalls are now better known.

The next phase of the study is to permit the engineers to observe and document the existing physical conditions of the seawall foundations, and to establish representative samplings of existing conditions. The detailed investigation will be conducted at selected sites in limited areas on both the landside and the seaward side of the seawalls. Geotechnical sampling along the seawalls will be taken during this phase.

Submitted is a full report on the historic research phase of the seawall study for the City of Charleston, South Carolina.

Executive Summary Phase III – The Detailed Investigation Phase

The detailed investigation phase included performing a geotechnical investigation, digging observation pits, and utilizing necessary investigation procedures to determine the geometry, construction materials, and condition of the seawalls and their supporting foundations.

For the stone masonry portion of the High Battery seawall, the detailed investigation was conducted at only one location along the landside seawall. The investigation of the Low Battery seawall was conducted at three landside sites and their adjacent seaward site locations. The concrete extension of

the High Battery seawall was investigated at one landside site and at one nearly adjacent seaward location. The Marina seawall was examined at one landside and its adjacent seaward site.

Detailed investigation of the seawalls surrounding Colonial Lake was not included in this scope of work.

In April 2003, Soil Consultants, Inc. performed a geotechnical investigation in seven locations along the seawalls. Two borings were taken in the right-of-way of East Battery adjacent to the High Battery seawall. Three borings were taken in the right-of-way of Murray Boulevard adjacent to the Low Battery seawall, and two borings were performed in the parking area adjacent to the Marina seawall.

Given the very close proximity of the borings to Charleston harbor and the magnitude of the tidal range, minimal amount of ground water fluctuation in all of the seven borings was quite surprising.

The geotechnical investigation clearly dispelled the notion that the level of the ground water on the landside of the seawall fluctuates synchronously with the level of the tide and through an equivalent range.

Furthermore, both the elevations and the minor fluctuations of the ground water table indicated that the timber platform supporting the Low Battery seawall was continuously below the ground water table. With constant submergence below the ground water, the timber structures supporting the concrete seawall would have been preserved.

During the detailed investigation phase, observation pits were planned and carefully excavated at selected sites along the lengths of the various seawalls. Typically, a landside observation pit was excavated directly across the seawall structure from where a seaward observation pit had been excavated.

The site locations selected for the detailed investigation were considered to be representative of the range of deterioration within major sections of the seawalls. The sites were not selected just to exemplify the worst levels of deterioration.

The general results of the detailed investigation are found in the following report.

With Phase III of the seawall study complete, the construction materials, the geometry, and the conditions of the seawalls are now better known.

The next phase of the study is the comprehensive engineering analysis and proposed repair recommendations. The goals of the analysis include determining conceptual repair and/or stabilization options, estimating construction costs, and establishing priorities for repairs and stabilization of the seawalls.

Submitted is a full report on the detailed investigation phase of the seawall study for the City of Charleston, South Carolina.

Executive Summary Phase IV – The Comprehensive Engineering Analysis and Repair Recommendation Phase

The recommended sequence of priority among the historic seawalls for maintenance is as follows:

1. The Concrete Extension of the High Battery Seawall,
2. The Low Battery Seawall,
3. The Stone Masonry Portion of the High Battery Seawall,
4. The Marina Seawall.

The Concrete Extension of the High Battery Seawall

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the seaward seawall and the concrete walkway are in poor condition. The seaward seawall portion and the concrete walkway portion appear to be at or near the end of their respective service lives. In its present deteriorated condition, it seems doubtful that the concrete extension of the High Battery seawall could successfully withstand the direct onslaught of a major hurricane without substantial damage. If this seawall should be significantly breached during a major hurricane, hurricane driven waves could propel flood waters well into the southeastern portion of the peninsula.

In consideration of the extent, nature, and location of the deterioration it appears that the replacement alternative would be both the more economic and practical choice.

The conceptual estimate of total engineering design and construction costs is $1,800,000.

The Low Battery Seawall

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the exposed portions of the seaward face of the concrete seawall range from poor condition to fair condition. The tilting of the sidewalk is primarily a long term soil settlement issue, not a direct seawall issue. The Low Battery seawall appears to be well along in its service life span. It is not possible, however, to realistically predict the number of years remaining in its service life span with any degree of certainty. The seawall is immediately adjacent to Murray Boulevard with an extensive neighborhood of homes beyond. Its length and location make the Low Battery seawall a very significant structure for protecting the peninsula. Currently, the Low Battery seawall

is in need of significant amounts of reactive preventive maintenance and lesser amounts of corrective maintenance to extend the service life span to its full potential.

The conceptual estimate of total engineering design and construction costs is $5,500,000.

The Stone Masonry Portion of the High Battery Seawall

On top of the seawall, the flagstone walkway is considered to be overall in poor condition. The flagstone walkway includes the integration of the flagstones, mortar joints between individual stones, the supporting underlying fill, and the bearing of the ends of the flagstones on the seaward seawall and the landside seawall. The deteriorated support and joint conditions for the stones reduce the safety and comfort of the walkway. The stone masonry portion of the High Battery seawall warrants special care. The historic seawall structure commemorates Charleston's history to residents and visitors alike. The walkway portion of the seawall structure and the seaward face of the seaward seawall are in considerable need of reactive preventive maintenance and prescribed preventive maintenance. The proposed preventive maintenance will not substantially increase the seawall's ability to successfully withstand major hurricanes. Rather, the proposed preventive maintenance is focused on increasing pedestrian safety and comfort as well as addressing concerns regarding the ongoing erosion of the mortar joints along the seaward face of the seawall.

The conceptual estimate of total engineering design and construction costs is $ 800,000.

The Marina Seawall

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the seawall is in poor condition. The seawall's ability to resist overturning earth pressures and lateral loads due to high wind and wave action is seriously compromised. The seawall appears to be well past its effective service life span and in some places it appears to be approaching the end of its physical life span. The Marina seawall is in a relatively protected location when compared to the more exposed locations of the High Battery seawall and the Low Battery seawall. However, in its present deteriorated condition, the tabby seawall provides very limited protection to the adjacent parking lot and a neighboring three story building. To alter this, it is envisioned that a secondary seawall would be constructed on the landside of the existing seawall. It is also envisioned that the holes, cracks, and crevices in the existing deteriorated tabby concrete seawall be repaired with the historic formulation for tabby concrete. No repair would be made to the deteriorated timber structure integral to the tabby concrete seawall. Nevertheless, the exterior face of the seawall exposed to view would be consistent with its original historic character.

The conceptual estimate of total engineering design and construction costs is $ 1,200,000.

Combined Conceptual Estimate

For the concrete extension of the High Battery seawall, the Low Battery seawall, the stone masonry portion of the High Battery seawall, and the Marina seawall the combined conceptual estimate of total engineering design and construction costs is $ 9,300,000.

Submitted is a full report on the comprehensive engineering analysis and repair recommendation phase of the seawall study for the City of Charleston, South Carolina.

AUTHORITY

On August 7, 2002, the City of Charleston, South Carolina retained Cummings & McCrady, Inc. of Charleston, South Carolina as the general consultant for the study and repair of the city's various historic seawalls. Cummings & McCrady, Inc. subsequently retained local consulting engineering firms to provide specialized expertise including The Sheridan Corporation for structural engineering of waterfront structures, 4SE, Inc. for structural engineering of historic structures, Soil Consultants, Inc. for geotechnical engineering, and Forsberg Engineering and Surveying, Inc. for topographic surveying.

PURPOSE OF THE SEAWALL STUDY

The historic seawalls are a defining feature of Charleston. The seawalls are showing definite signs of deterioration from long-term exposure to the elements. The first step in the seawall repair project is to conduct a comprehensive study of the existing conditions of the seawalls. The primary purpose of the study is to establish the requirements necessary for the seawalls to be repaired. The secondary purpose is to establish baseline conditions for use in any future monitoring of the seawalls' structural conditions.

The historic seawalls included in this study are the High Battery, the Low Battery, the seawalls surrounding Colonial Lake, and the portion of The City Marina seawall immediately adjacent to Lockwood Boulevard.

The seawall study has the following objectives:

1. To document and assess the general existing conditions of the seawalls;
2. To provide conceptual repair and/or stabilization options and recommendations;
3. To prepare estimated costs of construction; and
4. To set priorities for the repairs.

Thus, the seawall study becomes the basis on which subsequent cost/benefit analyses can be performed, and if determined to be a viable project, the starting point for the far more comprehensive design phase.

SCOPE AND LIMITATIONS OF THE SEAWALL STUDY

The seawall study is subdivided into the following phases:

Phase I is the general condition assessment phase.

Phase II is the historic research phase.
Phase III is the detailed investigation phase.
Phase IV is the comprehensive engineering analysis and repair recommendation phase.

Phase I was not planned to be an exhaustive inspection with an item-by-item inventory of defects, but rather to be general in nature supplying an assessment of the existing visible structural conditions. The inspection includes "walk-by" and "float-by" visual inspections of the seawalls. Photographs were taken to document representative conditions. This simple type of inspection does not involve the cleaning of marine growth from any structural elements. The purpose of the general condition assessment is to detect any obvious major damage or deterioration and to provide initial data for the subsequent detailed investigation phase. This general condition assessment is a non-mathematical structural assessment of the apparent structural conditions. The "float-by" inspection was conducted at low tide so as to provide the maximum visible exposure of the face of the seawalls.

Phase II of the seawall study includes research into the original engineering design and construction of the historic seawalls, since there are no known as-built drawings of these seawalls on file with the City of Charleston Engineer's Office. The intent of the research phase is to uncover written and/or graphical information about the seawalls and their foundations. The purpose is to provide as much structural background information about the seawalls as possible in order to supplement and fine-tune the planning for the subsequent detailed investigation phase. The research will be conducted at local historical libraries and other public archives.

Phase III of the seawall study includes the detailed investigation of the seawalls and will be conducted at selected sites in limited areas on both the landside and the seaward side of the seawalls. It is anticipated that in these locations, the earth around and under the seawall foundations will be carefully excavated to permit the engineers to observe and document the existing physical conditions of the seawall foundations, and to establish representative samplings of existing conditions. Once the observations are complete, the excavations will be immediately backfilled to limit the risk of possible foundation instability. The detailed investigation includes a topographic survey and geotechnical sampling along the seawalls.

Phase IV of the seawall study includes the engineering analysis of the condition of the seawalls based on the information obtained during the previous three phases of the study. Considerations include the nature, extent, and severity of the seawalls' deterioration in conjunction with any documentation of the original design and construction of the seawalls. The goals of the analysis include establishing priorities for repairs and stabilization of the seawalls, determining conceptual repair and/or stabilization options, estimating construction costs, and providing subsequent recommendations.

The scope of services stipulates that the seawalls of the High Battery, the Low Battery, and the Marina seawall immediately adjacent to Lockwood Drive shall be included in all of the four phases of the study. The seawalls surrounding Colonial Lake shall be included in the first two phases of study only.

After each phase of the study is completed, an interim report will be submitted based on the results of that phase of the study. When the entire study is complete, a comprehensive final report will be issued.

TERMINOLOGY

Seawall, bulkhead, wharf, quay, and quay wall are types of waterfront structures and their defined differences are subtle.

A seawall is a soil retaining and an armoring structure whose purpose is to defend the shoreline against wave and water erosion. Seawalls are forms of shore protection and are not intended for use as berthing facilities. Seawalls typically are massive coastal structures and are typically constructed of a variety of materials including rubble-mounds, granite masonry, or reinforced concrete. They may be supplemented with steel or concrete sheet pile driven into the ground.

A bulkhead is a flexible soil retaining wall structure typically comprised of vertically spanning sheet piles or other flexural members. Bulkheads typically derive their stability, in part, through passive earth pressures. Passive earth pressures act on the embedded portion of the structure between the lower ground line and the embedded tip of the structure. In some designs, bulkheads may have an additional lateral restraint system in the form of structural tie backs and piles. Bulkheads establish and maintain elevated grades along shore lines in relatively sheltered areas not subject to appreciable wave attack. Bulkheads are commonly used in conjunction with the berthing of vessels. In these instances, it is also known as a bulkhead wharf.

A wharf is generally an open deck parallel and adjacent to the shore line. It may or may not be contiguous with the shore. When the wharf is connected to the shore along its full length and a retaining wall is used to contain the upland fill placed behind the wharf, it is called a marginal wharf. The retaining wall portion is called the bulkhead. A wharf completely constructed with solid fill behind vertical walls and without an open structure may also be referred to as a quay. The quay wall is the seaward face of the quay. Thus, the quay wall is also an earth retaining structure with the functions of providing shore protection against light to moderate wave action and providing a berthing face for vessels. In these instances, the quay wall's multiple functions are similar to a bulkhead's. However, a quay wall is utilized when the overall height requirements or wave severity exceed the practical capabilities of typical bulkhead construction.

The basic functions of these various types of waterfront structures can become nonspecific overtime. For example, a waterfront structure that was originally constructed to provide a berthing face for vessels may no longer be utilized for that purpose. What was originally constructed and accurately described as a "quay wall" could, at present, more accurately be described as a "seawall".

For simplification in this report, the term "seawall" will be used to describe the structures generically

even though the structures might incorporate predominant historic features and be more accurately and narrowly defined as a wharf or a quay wall.

SEAWALL DESCRIPTIONS

The High Battery

The High Battery seawall extends approximately 3/10 of a mile in length in the general north-south direction along the west bank of the Cooper River. The street named East Battery is parallel and adjacent to the High Battery seawall on its land side. The northern end of the High Battery seawall is located approximately 200 feet north of the intersection of Water Street and East Battery. From this location, the High Battery seawall extends to the south and terminates just past the southeastern tip of White Point Gardens. At its southern end, the High Battery seawall curves approximately 90 degrees to the west and intersects with the eastern end of the Low Battery seawall. Similarly, near the southeastern tip of White Point Gardens, the parallel East Battery turns approximately 90 degrees to the west where it intersects with the eastern end of Murray Boulevard.

The present High Battery seawall was constructed utilizing two parallel seawalls located approximately 13 1/2 feet apart out-to-out. Fill was placed between the parallel seawalls and capped over with a walkway. Most of the seaward seawall is constructed with a stone facing. The very southern end of the seaward seawall where it curves to the west to intersect with the Low Battery is constructed of concrete.

The stones in the facing of the seaward seawall have been laid in a running bond pattern. The stones generally vary between approximately 18 inches to 20 inches in height and between 5 feet to 10 feet in length. The abutting faces of the stones are cut square and the joints are generally straight and of consistent width. In contrast, the exposed faces of the stones are rough and unfinished. The landside seawall appears to be constructed of brick masonry with a stucco coating applied that has been scored to imitate stone masonry construction. The majority of the landside seawall is obscured from view from the street by a hedge of oleanders. A slate coping, approximately 5 inches in depth, is situated on the top of the landside seawall and extends the length of the flagstone walkway.

The flagstone walkway extends nearly the entire length of the High Battery. The flagstones are approximately rectangular in shape and generally similar in size. The “typical” flagstone is approximately 9 feet in length and 4 feet in width. The flagstones are placed lengthwise between the stone copings of the seaward and landside seawalls to form the walkway surface. The tops of the flagstones are generally level with the tops of the seaward and landside seawalls. In this arrangement, flagstones have been placed side-by-side with approximately 1/2 inch to 1 ½ inches wide joints between the adjacent sections of flagstone. The joints had been filled with mortar to provide a continuous walking surface. Near the southern end of the High Battery, a concrete

walkway replaces the flagstone walkway.

During low tide, the shore line becomes exposed alongside and immediately adjacent to the entire length of the seaward seawall. When viewed from the harbor at time of low tide, the High Battery has approximately 10 feet of seawall height exposed for most of its length. The extreme northern end of the seawall has only 4 feet of wall height exposed to view because of a built-up sand beach.

On the land side, the height of the landside seawall exposed to view varies between approximately 2 ½ feet to 4 ½ feet. A railing extends the length of the High Battery along the seaward side and incorporates concrete pedestals, each approximately 3 ½ feet in height, and located approximately every 10 ½ feet on center. Three rows of plastic encased steel pipe railing extend between the pedestals. Concrete stairs are located at the extreme southern end of the High Battery and approximately every 200 feet along the landside seawall for convenient pedestrian access.

The Low Battery

The Low Battery seawall extends approximately 9/10 of a mile in length in the general east-west direction along the north bank of the Ashley River. At its eastern end near the southeastern tip of White Point Gardens, the Low Battery seawall intersects with the High Battery seawall. At this location, concrete stairs provide pedestrian access up the approximately 3 1/2 feet from the top of the Low Battery sidewalk to the High Battery walkway. The concrete sidewalk is parallel and immediately adjacent to the Low Battery seawall and extends the entire length of the seawall on its landside. Parallel and immediately adjacent to the sidewalk is Murray Boulevard. Murray Boulevard extends the entire length of the Low Battery.

Approximately halfway along its length from its eastern end, the Low Battery seawall curves and then continues on in a more northwesterly direction for the remainder of its length. The western end of the Low Battery seawall is in the vicinity of the Coast Guard Base and at the intersection of Tradd Street with Murray Boulevard.

Unlike the dual seawall configuration of the High Battery, the Low Battery was constructed utilizing a single seawall constructed with reinforced concrete. It is our understanding that the concrete seawall is supported on timber piles. It is also our understanding that at the base of the seaward side of the seawall there is a continuous array of concrete slabs for the protection of an underlying array of timber sheet piles.

During low tide, the shore line becomes exposed alongside and immediately adjacent to the entire length of the Low Battery seawall. When viewed from the Ashley River at time of low tide, the Low Battery seawall has approximately 6 feet to 7 feet of wall height exposed to view.

A railing extends the length of the Low Battery seawall. The railing from the eastern end at White Point Garden to King Street incorporates a mixture of granite and concrete pedestals on top of a 10-

inch high coping. The pedestals are each 3 1/2 feet in height and located approximately every 10 1/2 feet on center. Three rows of plastic encased steel pipe railing extend between the pedestals. The railing system along the Low Battery seawall from King Street to the western end at Tradd Street is slightly different. Along this stretch, the railing incorporates concrete pedestals approximately 30 inches in height located approximately 10 feet on center along the top of the concrete coping. Two rows of plastic encased steel pipe railing extend between the pedestals.

Marina Seawall

The portion of the marina seawall included in this study is located on the north bank of the Ashley River near the northeast side of the present-day City Marina and in the vicinity of the outflow from the old yacht basin. When viewed in plan, the marina seawall is basically "L" shaped. The seawall begins at Lockwood Boulevard and extends away, nearly perpendicular to the street in the general south-west direction, for approximately 200 feet. The wall then turns approximately 90 degrees to the northwest, extends approximately another 150 feet, and finally ends near the Variety Store Restaurant building. The seawall appears to have been constructed with tabby concrete and is approximately three feet in overall height. At time of low tide, a mudflat is exposed and extends hundreds of feet from the base of the seawall to the waters edge. The mean high water level is just below the top of the wall. During astronomically or meteorologically higher than normal tides, the river level overtops the seawall. On the land side of the seawall, the earth fill once leveled with the top of the seawall has eroded away.

Seawalls Surrounding Colonial Lake

When viewed in plan, Colonial Lake is approximately rectangular in shape and is completely surrounded by perimeter seawalls. The seawalls surrounding Colonial Lake function more like retaining walls for the earth on the landside than true "seawalls". In the longitudinal direction of the lake, approximately the north-south direction, the seawalls on each side of the lake are approximately 800 feet in length. In the lateral direction of the lake, approximately the east-west direction, the seawalls on each end of the lake are approximately 450 feet in length. The lake is bounded by Ashley Avenue to the west, Beaufain Street to the north, Rutledge Avenue to the east, and Broad Street to the South. On the land side, a perimeter sidewalk is immediately adjacent to the top of the seawall.

On the west side of the Colonial Lake, a single 42-inch diameter subterranean drainage pipe leads to the Ashley River and provides some flushing action with each tidal cycle. Colonial Lake apparently fluctuates approximately one foot in height during a tidal cycle. The overall wall height is approximately four feet with the exception of a slight rise near the north end of the wall along Ashley Avenue. At this location, the overall wall height rises to approximately 5 feet.

At time of low water, the water depth of the lake is approximately six inches immediately alongside

the seawall, and the height of the seawall exposed to view is approximately 3 1/2 feet. The seawalls surrounding the lake appear to be constructed predominately of tabby concrete with a two to three inch thick facing of more modern Portland cement stucco.

PHASE I - THE GENERAL CONDITION ASSESSMENT PHASE

ASSESSMENT CRITERIA

The categories used in this report for assessing the structural condition of the waterfront structures are "Good", "Fair", and "Poor". For concrete, these overall evaluation categories are based on the guidelines outlined in the reference manual Protection, Inspection, and Maintenance of Marine Structures by Pile Buck, Inc., 1990, and the Bridge Inspector's Manual/90 by the Federal Highway Administration, July 1991. For stone masonry, the evaluation will parallel the concrete criteria.

In general, "Good" condition means that the structure only has minor defects; "Fair" condition means the structure has moderate defects; and "Poor" condition means that the structure has major or severe defects. When it is difficult to put the structure completely into one category, a term such as "Fair to Poor" condition is used to signify such a marginal call. It should be recognized that in borderline cases, two different engineers could easily rate the same structure in different categories. The evaluation of the point at which a "minor" defect becomes a "moderate" defect, or the point at which enough "moderate" defects become a "major" defect is subjective. Even a condition assessment of a measurable section loss is subjective based on what is considered a "permissible" section loss. In this report, we feel the various overall condition assessments of the structures have been made with a conservative viewpoint.

For the longer seawalls, if conditions vary significantly within their length, the condition assessment will be zoned accordingly with reference to a specific street address, the nearest cross street, or other prominent landmark.

It needs to be emphasized that the condition assessment is only preliminary. The condition assessment is not an evaluation of structural capacity. The determination of such would necessitate a structural analysis.

For this preliminary inspection report, the condition assessment criteria of the concrete structures are as follows:

The structure was assessed to be in **"Good"** condition when concrete material remained hard and the defects were minor, such as occasional minor cracks, minor pits, or occasional shallow surface spalls in which the course aggregate was exposed but not the reinforcing bars.

The structure was assessed to be in **"Fair"** condition when the defects were moderate, such as moderate sized spalls with minor corrosion of exposed reinforcing bars, rust stains tracing along the path of reinforcing bars with or without visible cracking to the concrete, or surface disintegration to one inch due to weathering or abrasion.

The structure was assessed to be in **"Poor"** condition when the defects were major, such as a 10% to 15% loss of section, exposed and badly corroded main reinforcing bars, badly corroded reinforcing ties, deep wide cracks along reinforcing bars, or large spalls six inches or more in width or length.

There are no objective assessment criteria for stone masonry structures available in the Protection, Inspection, and Maintenance of Marine Structures or the Bridge Inspector's Manual/90. However, the inspection of stone masonry structures is somewhat similar to that of concrete structures.

The inspection of a stone masonry seawall may include examining for joint leaks; settlement that has occurred due to loadings, soil subsidence, or foundation failure; stones that have been displaced by wave action; and scouring or undercutting that has occurred due to sea action. The primary forms of deterioration of the stone blocks themselves are as follows:

1. weathering – its hard surface degenerates into small granules giving stones a smooth rounded look,
2. spalling – small pieces of rock break out or chip away, and
3. splitting – seams or cracks open up in rocks eventually breaking them into smaller pieces.

For this report, the condition assessment criteria of the stone masonry structures will parallel the assessment criteria for concrete structures, but on a more subjective basis, taking into account all of the criteria mentioned in the paragraph above.

The structure was assessed to be in **"Good"** condition when the defects were minor, such as occasional minor cracks, occasional shallow surface spalls, or minor loss of mortar in the joints, little settlement and no significant stone displacement.

The structure was assessed to be in **"Fair"** condition when the defects were moderate, such as more frequent minor or moderate cracks, more frequent and larger surface spalls, or moderate amounts of loss of mortar in the joints, more settlement and minor stone displacement.

The structure was assessed to be in **"Poor"** condition when the defects were major, such as 10% to 15% loss of stone cross section, large spalls of six inches or more in length or width, cracks or pits in the stone, or major amounts of loss of mortar in the joints, significant settlement or serious stone displacement.

GENERAL RESULTS OF THE "WALK-BY" AND "FLOAT-BY" INSPECTION

On November 7, November 15, and November 22, 2002, Mr. John Sheridan IV, P.E. from the Sheridan Corporation, and Mr. Craig Bennett Jr., P.E. from 4SE, Inc. performed the "walk-by" and "float-by" visual inspection of the specified seawalls. Mr. Jerome English, AIA, project manager from Cummings & McCrady, Inc. was intermittently present during various stages of the inspection.

High Battery Seawall

GENERAL CONDITIONS

The "walk-by" portion of the visual inspection was conducted from north to south along the flagstone walkway of the High Battery seawall. The flagstone walkway sometimes provided a "wobbly" sensation, particularly if one's weighted foot was placed near the edge of the flagstone. It is believed that the flagstones were to be supported by underlying fill placed between the seaward and landside seawalls. The fill has subsided over the intervening years and the loss of support has contributed to the "wobbly" effect. When a ruler was placed in the open joints between the flagstones, the stones were measured to be approximately 4 to 5 inches thick and the top of the fill varied between approximately 3 to 5 inches below the bottom surface of the flagstones. At present, the flagstones span between the seaward seawall and the landside seawall. It is possible that under certain conditions of heavy loadings, a stone could become overstressed and give way.

Much of the mortar originally placed between the adjacent sections of the flagstones no longer exists. The loss of support from the underlying fill probably contributed to the failure of the mortar joints. The longitudinal mortar joints between the flagstone walkway and the stone copings of the seaward and landside seawalls are also greatly deteriorated.

The top surface of the flagstone slabs exhibit a wide range of conditions. Some slabs are smooth and still provide a good walking surface while other flagstones exhibit significant amounts of surface scaling, creating potential tripping hazards for pedestrians. At the joints between adjacent slabs, there were fairly small variations in elevations between the tops of the slabs. Nevertheless, in some locations, the variations in the elevations between adjacent flagstone sections approached ½ inch creating additional potential tripping hazards. Some sections of the flagstones have previously cracked and were repaired by filling the cracks with mortar. At present, these previous crack repair attempts are in poor condition.

At the southern end of the High Battery, portions of the concrete walkway and the top portion of the landside seawall supporting the slabs have severely deteriorated. In other locations along the face of the landside seawall, there are vertical cracks varying between 1/8 inch and 1/4 inch in width.

The "float-by" inspection of the seaward seawall of the High Battery was conducted starting at the

north end of the seawall and progressing to the south. With the exception of the top one or two courses of the stone masonry, most of the mortar that was once present in the face of the joints has disintegrated. Regardless, the stones themselves show few signs of deterioration or dislocation. When impacted with a hammer during the inspection, the stones sounded solid, remained hard, and no chips or flakes of stone fell off. There were no indications of scaling on the faces of the stone. The lower two or three courses of stone within the tidal zone have become discolored with a brown hue and as expected, the stones have a covering of marine growth.

The majority of the seaward seawall of the High Battery indicates that high quality stone masonry workmanship was used during its construction. However, there are two areas of the seaward seawall which have curious variances in the stone masonry coursing. Progressing from north to south, the first variance is a "sag" in the vicinity of 25 East Battery and Atlantic Street and the second variance is a "rise" and then a "dip" near the southern end of the seawall. There is an additional minor "sag" near 19 East Battery.

The "sag" in the vicinity of 25 East Battery and Atlantic Street is a gentle "sag" in the coursing of the stone masonry. It is believed that unexpected differential settlement occurred to the seawall in this location. The "sag" is a couple of hundred feet in length with the lowest point occurring near 25 East Battery. The differential settlement was compensated with the placement of an additional row of "filler stones" just below the coping of the seawall and as a result, the top of the seawall remains essentially level. The additional row consists of relatively smaller stones. The additional stones vary between a minimum of less than 1 inch in height at the ends of the "sag" to approximately 6 inches in height at the most extreme part of the "sag". The additional stones are typically less than a foot in length. It is not known whether the additional course was placed during the original seawall construction or during a later repair.

The other variance in the coursing is an approximate 100-foot long "dip" near the seawall's southern end at White Point Gardens. The "dip" is also indicative of differential settlement and ends abruptly at the intersection with the concrete seawall. The top elevation of the stone seawall also slopes down slightly. In this zone, the stone masonry workmanship appears to be of a lower quality. The courses of the stone masonry are less regular and the stones vary considerably in height and width. An additional course of "filler" stones is located along the underside of the stone coping. The adjacent concrete section of the High Battery seawall is believed to be supported on a piled foundation. Though this section of the seawall suffers from significant deterioration, there are no indications of foundation settlement. Near the base of the seawall, holes appear in the concrete facing. The opening of one hole was approximately one square foot in area and a six foot rule could be extended approximately two feet into the opening. It is believed that this severe deterioration is occurring to the vertical concrete slabs located at the base of the pile supported seawall. It is also believed that these concrete slabs were intended to protect underlying timber sheet and support piles from marine boring organisms. There were no signs of any timber sheet pile behind the deteriorated concrete slabs.

OVERALL GENERAL CONDITION ASSESSMENT OF THE HIGH BATTERY SEAWALL

1. The flagstone walkway (includes the flagstones, the mortar joints between adjacent stones, and the "supporting" underlying fill) = **POOR.**

2. The individual flagstones = **GOOD to FAIR.**

3. The stone masonry portion of the seaward seawall (includes the stones and the mortar joints between adjacent stones) = **FAIR.**

4. The individual stones composing the seaward seawall = **GOOD.**

5. At the south end, the concrete portion of the seaward seawall = **POOR.**

6. At the south end, the concrete portion of the walkway = **POOR.**

7. The landside seawall = **FAIR.**

8. The railing (does not meet current safety standards) = **GOOD to FAIR**.

Low Battery Seawall

GENERAL CONDITIONS

The "walk-by" and the "float-by" inspections of the Low Battery Seawall were conducted from east to west. The visual inspections revealed fairly consistent patterns of deterioration but in varying degrees of severity.

From the eastern end at White Point Gardens to the vicinity of 14 Murray Boulevard, which is slightly west of the intersection with King Street, the "walk-by" inspection revealed the sidewalk to be level and in good condition with no apparent signs of settlement.

From the vicinity of 14 Murray Boulevard to the western end at Tradd Street, the sidewalk has two general settlement patterns. For description purposes, the concrete sidewalk along the length of the low battery consists of two sections of adjacent sidewalk paving. The seaward section of the sidewalk is immediately adjacent to the coping of the seawall and the street-side section of the sidewalk is immediately adjacent to the street.

From the vicinity of 14 Murray Boulevard to the vicinity of 50 Murray Boulevard, differential settlement seems to be occurring between the two adjacent sections of sidewalk. The seaward section of sidewalk has a slight slope downward toward the street. At the joint separating the two sections of sidewalk, the street-side section of sidewalk is approximately ½ inch lower than the seaward section and slopes steeply downward to the street and the curb. Apparently there have been unsuccessful repair attempts to lessen the abrupt change in elevation between the two sections of sidewalk. In the vicinity of 14 Murray Boulevard, the top of the sidewalk immediately adjacent to the coping is approximately 10 inches below the top of the coping.

From the vicinity of 50 Murray Boulevard to the western end of the seawall at Tradd Street, a seemingly different settlement pattern occurs. (The rebuilt level sidewalk between Limehouse Street and Council Street is the exception.) In this zone, both the seaward sections and the street-side sections have a consistent and relatively steep downward slope to the street level. In this zone, no sudden change in elevation occurs along the joint separating the seaward side and the street side of the sidewalk. Together, both sections appear to be rotating downward about the base of the coping. In the vicinity of 82 Murray Boulevard, the difference in elevation between the top of the sidewalk side immediately adjacent to the top of the seawall's coping is approximately 18 inches. Towards the western end of the seawall, there is a slight vertical waviness in the sidewalk when viewed along its length.

The concrete sidewalk shows additional distress near some of the storm drain inlets. In these locations, the sidewalk slabs are severely cracked from localized loss of subgrade support. It is suspected that cracks in the subterranean storm drain pipes are contributing to the erosion of the soils resulting in additional settlement of the sidewalk.

The "float-by" inspection was conducted around the time of low tide. The general patterns of deterioration are consistent with those anticipated for reinforced concrete approximately eighty years of age in a marine environment. Over the years, the salt laden environment has permeated the protective cover provided by the concrete. Consequently, in many locations, the steel reinforcing bars have corroded. The corrosion of the steel resulted in expansion, which caused the protective concrete to subsequently crack and spall and expose more of the steel reinforcing bars to the environment. This cycle of deterioration continues at an accelerating rate.

The quality of the concrete used in constructing the seawall between the eastern end at White Point Garden to King Street appears to be different from the quality of the concrete used for the remainder of the seawall. The seaward face of this eastern portion of the seawall appears more deteriorated. Where spalls have occurred, the exposed underlying concrete appears to be composed predominately of coarse aggregate with insufficient quantities of cement and fine aggregate. In general, the western portion of the Low Battery seawall appears to have had a higher quality of concrete than the portion of the Low Battery east of King Street.

Vertical expansion joints located approximately every 95 feet along the length of the seawall are in general poor condition. Much of the seaward face of the Low Battery seawall typically exhibits a

pattern of small, shallow indentions of approximately ½ inch to 1 inch in depth.

Between the eastern end of the seawall at White Point Garden to King Street, the length of the seawall coping exhibited very severe deterioration. Sections of concrete have spalled off to expose severely corroded steel reinforcing bars. The steel reinforcing bars appear to have had approximately one inch of protective concrete cover instead of the expected three inches of cover. Heavy rust stains bleeding from cracks parallel to the reinforcing bars are a common sight.

From King Street to Tradd Street, sections of the coping have been rebuilt and this pattern of severe deterioration occurs less frequently. The rebuilt sections of copings are easily detected by the differences in the concrete color. Corroded steel bolts, located approximately six to eight inches below these sections of coping extend outward from the face of the seawall. The bolts were probably used in a recent coping replacement project to anchor the concrete's temporary formwork.

When viewed from the river, portions of the seawall just below the coping exhibit prominent horizontal cracks approximately 1 inch in height. Streaks of rust emanate from these cracks and stain the face of the seawall.

Throughout the length of the Low Battery, the concrete railing pedestals had occasional spalls and rust stained cracks.

In random zones along the face of the seawall and at approximately two feet above the shore line, prominent cracks, approximately one inch to two inches in height, appear and extend horizontally between 30 and 50 feet in length. It is believed that this severe deterioration is occurring along the joint where a row of vertical concrete slabs originally connected with the bottom of the seawall's piled foundation. These concrete slabs were intended to protect underlying timber sheet and support piles from marine boring organisms.

In other zones along the length of the seawall, the joint between the seawall and the concrete slabs widened even more significantly, between several inches to over a foot in height. In these locations, it is observed that the concrete slab system has slipped downward from the pile supported seawall and the connecting steel reinforcing bars have become exposed. The exposed reinforcing bars are severely corroded. In these locations, a six-foot rule could be extended several feet into the opening along the face of the seawall. There were no signs of any timber sheet pile behind the deteriorated concrete slabs.

At locations where storm drain outlets project through the seawall, the level of deterioration to the seawall is particularly severe. The opening in the joints between the concrete slabs and the base of the seawall vary between several inches to a foot in height and extend along the seawall as prominent cracks.

CONCERNS

The concrete slabs were installed to protect the underlying interior timber sheet piles and the timber piles supporting the Low Battery seawall from marine borers. The occasional check where the integrity of the protective concrete slabs had been severely breached did not reveal any indications of underlying timber sheet piles remaining. If the salt water has had consistent long term contact with the sheet piles and the seawall support piles, then in all probability so have the marine borers.

The physical condition of the timber pilings supporting the Low Battery seawall and the timber sheet piles along its seaward face are very much a concern.

OVERALL GENERAL CONDITION ASSESSMENT OF THE LOW BATTERY SEAWALL

1. The concrete sidewalk (from the eastern end at White Point Gardens to King Street) = **GOOD.**

2. The concrete sidewalk (from King Street to Tradd Street and includes the concrete slabs and the settlement of the underlying subgrade) = **POOR.**

3. The concrete coping (from the eastern end at White Point Gardens to King Street) = **POOR.**

4. The concrete coping (from King Street to Tradd Street) = **VARIES between GOOD and POOR.**

5. The railing (does not meet current safety standards) = **FAIR**.

6. The exposed seaward face of the seawall (from the eastern end at White Point Gardens to King Street) = **POOR.**

7. The exposed seaward face of the seawall (from King Street to Tradd Street) = **FAIR to POOR.**

Marina Seawall

GENERAL CONDITIONS

The “float-by” inspection of the marina seawall was actually a “walk-by” inspection along the mud flat on its seaward side. The topside “walk-by” and the seaward “walk-by” revealed a seawall in a very severely deteriorated condition. The seawall was originally constructed of tabby shell concrete. The tabby concrete seawall has large cracks and holes that extend completely through the side of the

wall. Over time, the earth fill on the landside of the seawall has sifted through theses openings with the cyclic action of the tides. As a result, the earth backfill on the landside immediately adjacent to the seawall is sometimes two feet below the top of the wall in the vicinity of the larger holes. The appearance of a series of potholes overgrown with vegetation extends for much of the length of the seawall on its landside. Walking on the immediate landside of the seawall is hazardous.

Past attempts to repair the landside of the seawall by the placement of additional fill material in the holes followed by the indiscriminate dumping of concrete and asphalt on the top of the fill have not been a satisfactory long term solution.

OVERALL GENERAL CONDITION ASSESSMENT OF THE MARINA SEAWALL

1. The marina seawall = **POOR.**
2. The landside of the marina seawall = **POOR.**

Seawalls Surrounding Colonial Lake

GENERAL CONDITIONS

The "float-by" inspection of the seawalls surrounding Colonial Lake began on the Rutledge Avenue side of the lake at the stairs leading down to the water. The inspection continued in a clockwise direction around the lake. The inspection revealed that the seawalls in general have apparent significant deterioration to their "seaward" surface.

The seawalls appear to have been originally constructed of tabby concrete. Subsequently, the exposed surface of the tabby concrete above the high water line was coated with a 2 to 3 inch thick layer of Portland cement stucco. For the majority of the length of the seawalls surrounding Colonial Lake, this stucco coating has deteriorated significantly and large sections of the underlying tabby concrete structure of the seawall are exposed. The stucco coating located on the northwest side of the lake seawall along Rutledge Avenue to the north of Queen Street appears to be less deteriorated.

Either the stucco coating was never applied to the seawall in the tidal and underwater zone or it was applied and has since completely deteriorated. In general, the tabby concrete seawall structure in the tidal and underwater zone has deteriorated and spalls varying from one inch to several inches in depth are common. Less common within the tabby concrete seawall are deep crevices; one crevice was measured to be 20 inches in depth.

Above the tidal zone, where exposed by the loss of the stucco coating, the underlying tabby concrete seems to be in comparatively better condition. It reveals a rough and unfinished appearance with cracks and shallow spalls on the seaward face, but the deterioration is not as severe as within the tidal and underwater zone.

The seawall along the west side (Beaufain Street side) appears to have had major stability problems in the past. Three heavily corroded circular steel plates, similar to the type used as part of an anchor tie back system, are visible on the lake side of the seawall. Further evidence is the appearance of a bulge in this location of the wall in the direction of the lake when viewed from above. It is speculated that there is an underground tie-back system consisting of steel rods connecting the plates to anchor piles driven back towards Beaufain Street.

The west end of the south side, along Ashley Avenue exhibited a less severe bulge outward toward the lake, but there are no steel tie back plates present in this zone.

It appears that the top elevation of the seawalls on the east side of the lake (along Rutledge Avenue to the south of Queen Street) and the south side of the lake have been increased in the past. Along these zones of the seawalls, distinct horizontal construction joints of differing material are present. In addition, the top surface of each layer appears to have been finished as if it were at one time the exposed top surface of the seawall.

The "walk-by" inspection revealed that the adjacent sidewalks and the land side of the seawall are in apparent good condition and the perimeter of the lake is very popular recreational area.

OVERALL GENERAL CONDITION ASSESSMENT OF THE SEAWALLS SURROUNDING COLONIAL LAKE

1. The seawalls surrounding Colonial Lake = **POOR** to **FAIR.**
2. The landside of the seawalls surrounding Colonial Lake = **GOOD.**

GENERAL DISCUSSION

The condition assessment is only preliminary. The condition assessment is not an evaluation of structural capacity.

The purpose of the general condition assessment phase of the seawall study was to detect any obvious major damage or deterioration and to provide initial data for the subsequent detailed investigation phase. Given the length of time these historic seawalls have been exposed to the harsh marine environment, the nature and degree of the deterioration observed was generally consistent with what was anticipated.

With Phase I of the seawall study complete, the observed physical conditions of the historic seawalls are better known and provide the starting point for the subsequent phases of the study.

The next phase of the study is the historic research into the original engineering design and construction of the historic seawalls. The purpose is to provide as much structural background information about the seawalls as possible in order to supplement and fine-tune the planning for the subsequent detailed investigation phase.

Remaining are many questions to be answered before any meaningful conclusions can be made regarding repairs to the historic seawalls.

Herein, concludes the discussion of the general assessment phase of the seawall study for the City of Charleston, South Carolina.

PHASE II - THE HISTORIC RESEARCH PHASE

HISTORIC RESEARCH

The City Of Charleston Archives, The Charleston Library Society, and the South Carolina Room of the Charleston County Library were the archival sources of historic information utilized herein. A special note of appreciation must be extended to their helpful staffs.

The Charleston Historic Foundation, The South Carolina Historical Society, The Preservation Society of Charleston, The Charleston Museum, The South Carolina Department of Archives and History, The Hagley Museum, The South Carolina State Library, and the Thomas Cooper Library at the University of South Carolina were also contacted. They reportedly did not archive the very specific historical information sought.

THE CONSTRUCTION OF THE SEAWALLS

High Battery Seawall

The years 1804, 1820, 1854, 1885, and 1893 were particularly prominent in the civil engineering history of the High Battery seawall. In these years, the construction and reconstruction of the High Battery seawall was defined by the ongoing defense of the Charleston peninsula against shoreline invasion and storm destruction.

In 1804, a severe storm swept away the palmetto log seawall that extended along most of the

frontage of the present day High Battery seawall. Its replacement was a seawall built with stone ballast from arriving ships. It is believed that the designation of "The Battery" comes from the placement of guns along the crest of this still incomplete seawall during the war of 1812-1815. Eventually, the stone seawall was completed circa 1820.

During the "gale of 1854", the new stone seawall was severely damaged. The seawall was subsequently repaired and raised to its present height, approximately five feet above the elevation of the adjacent street, East Battery. The seawall then became known as the "High Battery."

In August 1885, a hurricane sent large waves sweeping over the "promenade" of the High Battery seawall displacing the small flagstones that formed the walkway pavement. As a result, several hundred feet of the walkway were destroyed in addition to significant damage to the landside seawall.

To prevent similar damage from occurring in the future, the small flagstones were replaced with large slabs of stone of sufficient length to extend completely across the width of the walkway. The new stones were described in the City of Charleston's Year Book 1886 as being "ten feet long and not less than four inches thick." Additionally, new cedar railing posts and new iron hand rails were installed replacing the previous all timber railing.

In 1886, a major earthquake occurred in the Charleston area and many public structures were severely damaged. Although a record of earthquake damage to public structures is included in the Year Book 1886, this account does not mention any earthquake damage occurring to the High Battery seawall.

In 1893, a major hurricane, with a reported thirteen foot tidal surge, impacted the Charleston peninsula. The saturating storm water eroded the earth supporting the large flagstones of the recently reconstructed walkway. Consequently, the large stones dropped and pushed over the landside seawall. With major damage occurring to the seawall again, great consideration was given to strengthening and rebuilding the High Battery seawall to better withstand major hurricanes.

The 1893 plan called for providing interior concrete walls to be located under the walkway to reinforce both the stone masonry seaward seawall and the foundation wall of the landside seawall. A concrete wall two feet thick at the base and one foot thick at the top was constructed contiguous with the interior face of the stone masonry seaward seawall for its entire length. Similarly, an approximately one and a half foot thick concrete foundation wall was constructed contiguous with the interior face of the brick masonry foundation wall of the landside seawall. The result was a composite foundation wall nearly three feet thick overall. Upon this foundation, the top six feet of the brick masonry landside seawall was completely rebuilt and widened to approximately three feet at the base and two feet at the top. For the 1893-1894 reconstruction, only stones and bricks were allowed to be placed between the newly strengthened seaward and landside seawalls as backfill. No earth backfill was allowed. The flagstones forming the walkway were reinstalled and hid from view any indication that the seawalls had ever been strengthened. The undersides of the flagstones were

supported not only by the underlying backfill of stones and bricks, but also by the seaward seawall's interior concrete wall and the masonry landside seawall. This reconstruction project was completed in April 1894. Additionally, four double and one single flight of stone stairs leading up to the "promenade" were constructed. New iron railing pedestals were installed replacing the timber railing pedestals.

The concrete portion of the High Battery seawall located at its extreme southern end was constructed as part of the "Boulevard Extension" project of 1917-1919. This concrete portion provided the closure section for the connection with the Low Battery seawall.

A copy of the 1893 repair plans obtained from the City of Charleston Archives is included in Appendix E.

Low Battery Seawall

A former seawall along the Ashley River extended from the High Battery seawall to the foot of King Street and established the southern boundary of White Point Garden. The seawall was constructed between 1838 and 1852 as part of the vision for White Point Garden. This seawall is now covered by earth fill and is located approximately 70 feet landward to the north of the present Low Battery seawall.

The present Low Battery seawall was constructed in two distinct phases as part of the overall "West End Improvement", a large land reclamation project. The seawall was designed to allow future extension. The vision was that the seawall would ultimately extend to the Ashley River Bridge.

"The Boulevard" was the first phase of the "West End Improvement" and was constructed between 1909 and 1911. During this phase, the seawall was constructed from the foot of King Street to the west end of Tradd Street. For much of its length, the seawall was constructed well seaward to the south of the then existing high tide shoreline. The forty-seven acre area between the new seawall and the previous high tide shoreline was subsequently filled with material dredged from the Ashley River. What had previously been "water lots" were then sold as building sites for new homes. Murray Boulevard was subsequently constructed along and adjacent to the seawall.

The contract for this first phase was awarded to McLean Contracting Company of Baltimore, Maryland. The contract was signed June 21, 1909 with the completion scheduled for January 21, 1911. During this phase 3,885 feet of seawall, entirely supported by a piled foundation, was constructed. The area enclosed by the perimeter of the seawall varied in elevation between approximately six feet below mean low water to six feet above mean low water. It was estimated that 667,000 cubic yards of dredged material, exclusive of shrinkage and settlement, were required to fill the entire 47 reclaimed acres to an elevation of 8.5 feet above mean low water. Approximately, 262 feet of timber barricades were constructed at the ends of the seawall and extended to the high land for retaining the placement of the fill material. The timber barricades were used not only to

save money in lieu of building concrete closure walls, but also with the expectation that the seawall would be extended both to the east and to the west in the near future.

The second phase, known as "The Boulevard Extension", was constructed between 1917 and 1919. The seawall was constructed between the foot of King Street to just past the southeastern tip of White Point Garden. Near its eastern end, the seawall was increased in height and curved northward forming the concrete extension at the south end of the High Battery seawall. Subsequently, the area between the new Low Battery seawall and the former seawall was filled with earth. The newly reclaimed land permitted the extension of Murray Boulevard to the southeastern tip of White Point Garden.

The contract for the second phase was awarded to Byran & Company of Jacksonville, Florida on March 29, 1917. The work included the construction of approximately 950 feet of seawall, the relocation of old granite railing pedestals from the former seawall to the new seawall, the extension of Meeting Street and the Church Street drains, the extension of the High Battery walkway, and the placement of fill in the reclaimed area behind the new seawall. Originally contracted to be completed in November 1917, the date for the completion was extended to August 1918, and later extended to June 1919. The project was finally completed December 1, 1919. Difficulties in obtaining labor and equipment due to the war in Europe were cited as the causes for the delays.

The design of the Low Battery seawall was thoroughly described in both of the City of Charleston's Year Book 1911 and the Year Book 1917. The 1917 edition reported that the same seawall design was used for both "The Boulevard" and "The Boulevard Extension", but slightly modified at the eastern end "where the wall was raised several feet to meet the elevated walkway of the High Battery". The excerpts from these resources are included in Appendix F.

The Low Battery seawall has three major foundation components which are the main concrete structure of the seawall, a timber platform supported by timber piles, and a timber sheet pile retaining wall covered with protective concrete slabs.

The main concrete structure of the seawall is certainly the most apparent component. Nevertheless, most of the seawall structure is hidden from view.

Record drawings obtained from the City of Charleston Archives provide interesting insight into the "West End Improvement" and are included in Appendix F.

Perhaps, the best graphic representation of the cross-section design through the Low Battery seawall appears in a drawing dated May 16, 1935, nearly sixteen years after the completion of "The Boulevard Extension." The cross-section is entitled "Tentative Section Proposed Sea Wall, Western Water Front, Charleston S.C. City Engineers Office". This cross-section design graphically correlates with great precision the dimensions and configurations as described in the Year Book 1911 and Year Book 1917.

Different designs of the seawall cross section were found in the archives. From the written description of the seawall design in the Year Book 1911 and Year Book 1917, only one design was apparently utilized. Perhaps the other cross-section designs were part of a design study or perhaps they were proposal drawings submitted by contractors as part of their bids. Giving some credence to the latter possibility, one cross-section design was submitted by Coastwise Dredging Company of Norfolk, Virginia and dated May 3, 1909, the bid date for the construction of "The Boulevard".

Two other archive drawings are of particular interest. The first drawing indicates a series of brace piles consisting of pairs of vertical and batter piles connected to the concrete portion of the seawall with a 1 ½ inch diameter rod. The City Engineer's approval is indicated with a note, "Brace piles, 40 at each curve in the wall". It is not certain if these brace piles were ever installed on the curves. There is no mention of these tie backs being used in either of the 1911 and the 1917 narrative. The plan was apparently submitted by McLean Contracting Company, the contractor on "The Boulevard."

The second drawing of particular interest is a survey map entitled "Map of the Boulevard Area Charleston, South Carolina, drawn to accompany Report on the Present Condition of the Work of Reclamation", dated May 3, 1911. The survey indicates the locations of failures in the newly constructed seawall. The most common failure was the outward (seaward) movement of the protective concrete veil. The survey also indicates that these failures in the concrete veil were repaired. It appears that the repair technique consisted of driving additional vertical piles at the toe of the seawall and placing cobbles and riprap at the base of the seawall.

Further extension of the Low Battery seawall westward from Tradd Street was never realized. However in 1989, an approximate 100-foot long permanent concrete closure section was constructed at the west end of Tradd Street on top of the apparent remains of the temporary timber barricade.

Marina Seawall

Research of this seawall yielded little specific design or construction information. It is believed that the seawall, with its proximity to the West Point Rice Mill, was constructed under private ownership as a wharf for the West Point Rice Mill facility.

Historic maps of peninsula Charleston were studied in an attempt to determine the period of time in which this seawall was constructed. The applicable maps are included in Appendix G. The seawall's prominent L-shaped configuration and its proximity to the historic West Point Rice Mill aid in its visual identification.

An 1855 map displays the prominent L-shaped configuration of the seawall at the West Point Rice Mill. A map dated 1844 shows only a portion of the L-shaped configuration. From these historic maps, it is believed that this seawall was completed in the time period between 1844 and 1855. The reviewer believes, because of the poor quality of the cartography of the 1844 map, that it is possible

that the wall existed in 1844.

Obtained from the City of Charleston Archives and included in Appendix H, is a copy of the "Plat of the West Point Rice Mill Property of the Estate of Thomas Bennett Lucas," dated March 1, 1860. The prominent L-shape configuration of the seawall is visibly adjacent to a creek.

Seawalls Surrounding Colonial Lake

No design or construction drawings of the seawalls surrounding Colonial Lake were found in the City of Charleston Archives. The descriptive civil engineering history of the seawalls surrounding Colonial Lake was obtained mostly from various City of Charleston, S.C. Year Books and excerpts are included in Appendix I.

In 1768 an act was passed to cut a canal from the southern end of Broad Street to the Ashley River and to reserve the vacant marshland on each side of the canal for use as a "Common" for Charles Town. The land set aside in perpetuity as a "Common" included the area between present day Beaufain Street, Rutledge Avenue, and Broad Street and extending west to shores of the Ashley River. Between 1768 and 1881, portions of the original tract were sold by the City.

In 1881, the City took steps to preserve the remainder of the property for its original purpose as a "Common". The vision was to create a "rival to White Point Garden." City Council appropriated funds for the improvement of the lake and grounds and a board of commissioners was appointed.

The lake, then known as "Rutledge Street Pond", was "a shallow body of water, its sides being boarded and the walks along the Broad, Rutledge and Beaufain Street oyster shells, with here and there boards, filling up the lower places."

In 1882, a concrete wall was constructed along three sides of the lake adjacent to Rutledge Avenue, Broad Street, and Beaufain Street. The concrete wall was 1,440 linear feet in length and circuited approximately two thirds of the perimeter of the lake. The concrete wall was three feet wide at the base, one foot-six inches wide at the top, and five feet- six inches high "from the foundation plank". It was estimated that 17,820 cubic feet of concrete wall was constructed in 1882. The material dredged from the bottom of the lake, in conjunction with fill from other sources, was placed adjacent to the landside of the concrete wall for the creation of walkways. A brick culvert thirty-six feet long with double gates four feet wide and five feet high was installed "as a means of flooding the lake and keeping up the salt water supply." Oak trees were planted and a shell walk constructed.

Because necessary additional appropriations were not immediately forthcoming, the complete enclosure of the lake with concrete walls was not finished until 1884.

The 1886 earthquake inflicted only minor damage to the concrete walls which were subsequently repaired. The open spaces around the lake served as a camp ground for those rendered homeless by

the earthquake.

In 1894, the concrete wall along Beaufain Street was increased in height. In 1896 the brick culvert on Ashley Avenue was replaced with two 20-inch terra cotta pipes and a new water gate was installed. In 1902, the seawall along Rutledge Avenue was increased in height and portions of the seaward face of the seawall were plastered.

PHASE II GENERAL CONCLUSIONS

Stone Masonry Portion of the High Battery Seawall

The reconstruction and strengthening of the stone masonry portion of the High Battery seawall in 1893 has clearly been a success. Since that time, the High Battery seawall has successfully weathered major hurricanes in 1911, 1940, 1959, and again in 1989 without any apparent significant damage.

For the stone masonry portion, the only apparent modification in approximately 110 years has been the replacement of the iron railing pedestals with concrete railing pedestals and replacement of the iron handrails with plastic coated steel pipe handrails. Excluding these minor modifications, the basic structure of the stone masonry portion of the High Battery seawall appears to be essentially unchanged since its last major reconstruction in 1893. The minimal maintenance requirements of the High Battery seawall over the last 110 years are a tribute to the superlative performance of the high quality stone masonry construction.

Nevertheless, 110 years of exposure to the harsh marine environment is a long time and, at present, the seawall shows signs of deterioration. The objective is to assure that the High Battery seawall continues to successfully withstand major hurricanes without significant damage.

Research confirms that the design of the concrete extension at the south end of the High Battery seawall is essentially the same as the design of the Low Battery seawall. Henceforth, further considerations of the concrete extension of the High Battery seawall will be included with those of the Low Battery seawall.

The Low Battery Seawall and the Concrete Extension of the High Battery Seawall

The main concrete structure of the seawall was constructed on and is supported by a timber platform deck that is supported by timber beams which are fastened to the supporting timber piles with steel bolts and spikes. The entire seaward face of the seawall is skirted with an array of timber sheet piles forming part of the retaining wall system to retain the landside fill. The timber sheet piles are covered with an array of protective concrete slabs. The deck of the timber platform is constructed

2’-6” above mean low water, essentially at the middle of the tidal range.

On the seaward side of the seawall, there are multiple locations where the protection provided by the concrete has been breached. Therefore, the timber sheet piles and the timber support piles may have been exposed to long term damage from marine borers.

On the immediate landside of the seawall, the level of the ground water may be fluctuating synchronous with the level of the tide and through an equivalent range. Subsequently, the timber support platform, the timber support beams, and the top portion of the support piles, would be in the zone of a fluctuating water table and potentially vulnerable to the effects of long term decay. Similarly, the steel bolts and spikes used in the connections of the timber support structure would be potentially vulnerable to the effects of long term corrosion.

The existing condition of the timber sheet piles, the timber platform deck, the timber support beams, the steel bolts, and the top portion of the timber support piles are very much a concern.

The Marina Seawall and the Seawalls Surrounding Colonial Lake

Research indicates that the Marina Seawall was completed in the time period between 1844 and 1855 and that the seawalls surrounding Colonial Lake were constructed between 1882 and 1884. Both of these seawalls were constructed with tabby concrete.

It appears that neither of these seawalls has been renovated since their original construction. At present, both of these seawalls display significant signs of deterioration.

Conclusion

The topographic survey of the High Battery seawall, the Low Battery seawall and the City Marina seawall were performed during this historic research phase. The topographic surveys are included in Appendix J.

With Phase II of the seawall study complete, the original engineering designs and/or the historic period of construction of seawalls are now better known.

The next phase of the study is to permit the engineers to observe and document the existing physical conditions of the seawall foundations, and to establish representative samplings of existing conditions. The detailed investigation will be conducted at selected sites in limited areas on both the landside and the seaward side of the seawalls. Geotechnical sampling along the seawalls will be taken during this phase.

Herein, concludes the discussion of the historic research phase of the seawall study for the City of

Charleston, South Carolina.

PHASE III – THE DETAILED INVESTIGATION PHASE

SCOPE OF WORK

The detailed investigation phase included performing a geotechnical investigation, digging observation pits, and utilizing necessary investigation procedures to determine the geometry, construction materials, and condition of the seawalls and their supporting foundations.

For the stone masonry portion of the High Battery seawall, the detailed investigation was conducted at only one location along the landside seawall. The investigation of the Low Battery seawall was conducted at three landside sites and their adjacent seaward site locations. The concrete extension of the High Battery seawall was investigated at one landside site and at one nearly adjacent seaward location. The Marina seawall was examined at one landside and its adjacent seaward site.

Detailed investigation of the seawalls surrounding Colonial Lake was not included in this scope of work.

GEOTECHNICAL INVESTIGATION

In April 2003, Soil Consultants, Inc. performed a geotechnical investigation in seven locations along the seawalls. Two borings were taken in the right-of-way of East Battery adjacent to the High Battery seawall. Three borings were taken in the right-of-way of Murray Boulevard adjacent to the Low Battery seawall, and two borings were performed in the parking area adjacent to the Marina seawall.

In each of the seven soil boring holes, fluctuations in the ground water level were monitored over a period of nine hours and correlated with the measured tide cycle during that period. The fluctuations of the ground water levels in the boring holes varied between approximately 0.1 to 0.4 feet. The range of tide in Charleston harbor during that period was approximately 4.8 feet.

The entire geotechnical report, boring logs, and ground water tables are included in Appendix K.

GEOTECHNICAL INVESTIGATION CONCLUSIONS

Given the very close proximity of the borings to Charleston harbor and the magnitude of the tidal range, minimal amount of ground water fluctuation in all of the seven borings was quite surprising.

The geotechnical investigation clearly dispelled the notion that the level of the ground water on the landside of the seawall fluctuates synchronously with the level of the tide and through an equivalent range.

Furthermore, both the elevations and the minor fluctuations of the ground water table indicated that the timber platform supporting the Low Battery seawall was continuously below the ground water table. With constant submergence below the ground water, the timber structures supporting the concrete seawall would have been preserved.

OBSERVATION PITS

During the detailed investigation phase, observation pits were planned and carefully excavated at selected sites along the lengths of the various seawalls. Following the excavation of an observation pit, the exposed seawall foundation was power washed. The engineers then had a better visual access to document the existing conditions of the seawall foundations. Each observation pit was excavated and subsequently backfilled within one tidal cycle. Typically, a landside observation pit was excavated directly across the seawall structure from where a seaward observation pit had been excavated.

Approval from Federal and State Regulatory Agencies was required for the implementation of the detailed seaside investigation. In May 2003, "The Joint Federal and State Application Form", in conjunction with the necessary maps and sketches, was submitted to the U.S. Army Corps of Engineers for permission to excavate/backfill the seaside observation pits.

Authorization from the U.S. Corps of Engineers to proceed was granted in November 2003.

ADDITIONAL RESOURCES

Salmons Dredging Corporation was contracted to provide the necessary equipment, manpower, and expertise for the excavation and subsequent backfilling of the observation pits on the seaward side of the various seawalls.

The City of Charleston Public Works Department coordinated any necessary permits and provided the necessary equipment, manpower, and expertise for the excavation and subsequent backfilling of the observation pits on the landside of the various seawalls.

The seaside detailed investigation was conducted during the month of January 2004, and the landside detailed investigation was primarily conducted during March 2004.

SPECIFIC LOCATIONS SELECTED

The site locations selected for the detailed investigation were considered to be representative of the range of deterioration within major sections of the seawalls. The sites were not selected just to exemplify the worst levels of deterioration.

The selected sites are defined by their topographic survey station numbers. For example, along the Low Battery seawall Station 9+73, near 84 Murray Boulevard, and Station 27+48, near 32 Murray Boulevard, are 973 feet and 2,748 feet respectively from the defined and marked "zero" (0+00) station near the seawall's intersection with Tradd Street. The topographic surveys of the inspected seawalls with their defining station numbers are included in Appendix J.

For the stone masonry portion of the High Battery seawall, the landside observation site is station 57+73, in the vicinity of 21 East Battery. Because the seaward seawall did not show significant structural deterioration, it was felt that it would be unnecessary to excavate a seaside observation pit at this time.

The three locations selected along the Low Battery seawall for both the landside and seaward investigations include:

1. Station 9+73, in the vicinity of 84 Murray Boulevard
2. Station 27+48, in the vicinity of 32 Murray Boulevard,
3. Station 43+50 near White Point Garden between the King Street and the extension of Meeting Street.

The landside and seaward observation sites for the concrete extension of the High Battery seawall are station 49+21 and station 49+24 respectively. These sites are near the intersection of East Battery and Murray Boulevard.

For the Marina Seawall, the landside and seaward observation site is station 2+38. This site is along the eastern leg of the seawall near its southern corner.

GENERAL RESULTS OF THE DETAILED INVESTIGATION

Stone Masonry Portion of the High Battery Seawall

The landside wall of the stone masonry portion of the High Battery seawall appears to have been constructed closely corresponding to the cross section sketch on page 1 of Appendix E.

Station 57+73

The conditions at this chosen site location correspond with the conditions displayed in the top photograph on page 7 in Appendix A. Photographs, field notes, and sketches from the detailed landside investigation are included in Appendix L.

Landside Investigation

An observation hole was excavated in the earth adjacent to the landside wall of the stone masonry portion of the High Battery Seawall. The landside investigation revealed a brick masonry wall approximately 11 feet in overall height from the top of the walkway to the bottom of the wall. The bottom 15 inches of the wall stepped out approximately 5 inches. The stucco coating on the brick wall terminated just below the ground surface. There were numerous hairline cracks in the stucco coating. However, these hairline cracks are not a sign of structural distress. A six inch high, continuous concrete band was visible approximately one foot below the bottom of the stucco coating. The location and height of the concrete band correspond closely with the cross section sketch on page 1 of Appendix E that details the construction of the concrete backup wall.

The mortar in the exposed brick masonry appeared to have a high oyster shell content. The mortar was soft but in fair condition. The brick in the masonry wall was hard and of approximately uniform size. Every other course of brick had been laid perpendicular to the previous course in an English (Cross) Bond pattern to provide mechanical bonding between wythes.

Two layers of timber planks, laid perpendicular to each other, extend under the base of the brick wall. It appears that these timber planks were intended to form a keyed platform for the bottom course of brick. The upper timber plank or “key” was approximately 13 inches wide and 3 inches thick (high) and extended perpendicular into the face of the wall at its base. The top and sides of the timber are in contact with the brick wall. Below and supporting the “key” timber, a three inches thick (high) timber layer extends parallel to the face of the wall in the approximate north-south direction. The two layers of timbers planks are situated below the ground water level and are considered to be in overall good condition. The timbers exhibited firm resistance to the point of the probe.

Beneath the lower layer of timber planks there were at least two layers of timber logs. The two layers of logs crossed perpendicular with each other. The top layer of logs extended in the north-south direction parallel to the face of the wall. The lower layer of logs extended in the east-west direction perpendicular to the face of the wall. The voids between the layers of the crossing logs were filled with brick rubble.

The logs in the top layer were approximately 6 to 8 inches in diameter. The bottom layer of logs had

diameters of approximately 12 inches and were immediately adjacent to each other, forming a nearly solid mat. The logs were in good condition, though slightly "spongy" in their outer perimeter.

Low Battery Seawall

The Low Battery seawall generally appears to have been constructed to the dimensions and configuration set forth in the archival design sketch on page 1 of Appendix F. Photographs, field notes, and sketches from the detailed investigation of the Low Battery seawall are included in Appendix M.

Station 9+73
(in the vicinity of 84 Murray Boulevard)

The site location corresponds with the photographs on pages 30-32 in Appendix B. The prominent sign of structural deterioration on the seaward side is the wide, horizontal crack located approximately six feet below the coping. The crack is approximately 95 feet long and extends between the vertical expansion joints in the seawall face.

Seaside Investigation

On the seaward face, in the zone below the horizontal crack and above the mud line, the concrete was generally of poor quality. In this zone, the concrete was soft, deeply fissured, broken, and easily removed using pneumatic hand tools.

Removing broken sections of the concrete around an existing hole in the face of the seawall resulted in a cavity with an opening approximately 6 feet wide and varying in height between 1 to 2 feet. The floor of the cavity extended inward approximately 2 ½ feet from the seaward face. The ceiling of the cavity sloped irregularly downward from the seaward opening. Further towards the interior of the seawall and above the ceiling of the cavity, the concrete was substantially harder and could not easily be removed with pneumatic hand tools.

The investigation revealed that the prominent horizontal crack was not positioned along the joint between the base of the pile supported cast-in-place seawall and the veil of protective precast concrete slabs. Instead, the long horizontal crack was positioned several feet above this joint within the concrete mass of the pile supported seawall.

Four batter piles and one vertical pile in the seaward row of support piles were exposed by the cavity opening. The tops of these timber support piles were not in contact with the exposed underside of the concrete seawall. In fact, the underside of the hard concrete seawall was several inches above the tops of these seaward support piles. Considering the deep crevices in this localized area of the

seaward face of the concrete seawall, there was remarkably little marine borer damage to these five exposed timber piles.

With the removal of the riprap stones at the mud line and the subsequent power washing of the seawall face, the joint between the base of the pile supported seawall and the array of protective concrete slabs could clearly be viewed. The portions of the steel reinforcing bars that had extended from the concrete protective veil into the base of the seawall were severely corroded. Nevertheless, the top edges of the exposed concrete veil sections and the timber sheet piles appeared intact and in good condition.

Consistent with the archival sketch on page 1 of Appendix F, a pair of exposed 5” wide x 12” deep timbers extended almost perpendicular to the face of the seawall. The timbers were bolted to the exposed vertical pile and its adjacent seaward batter pile. Approximately twelve inch deep by two inch wide notches had been cut into the sides of the piles to provide partial end support for the 5 inch wide timbers.

The steel bolts, nuts, and washers used in the connection of the 5” x 12” timber framing to the support piles were severely corroded. The ends of the bolted connections were heavily encrusted by a ferrous compound formed by the chemical decomposition of the bolts, washers, and nuts. The interior of the lengths of the bolts had corroded deeply inward from the ends. The remaining cross-section of a bolt was more comparable to a thin walled pipe than to a solid round rod. The shaft of a screw driver could be inserted into the ends of the bolts and pushed several inches into the interior length of the bolt.

Inconsistent with the archival sketch on page 1 of Appendix F, the seaward edge of the timber deck platform within this excavated cavity terminated approximately 2 1/2 feet landward from the seaward face of the seawall. The archival sketch clearly indicates that the timber deck platform was to extend to the seaward face of the seawall to provide support for the full width of the base of the cast-in-place concrete seawall.

The wood samples removed from the various exposed timbers appeared well preserved. However, for a depth of one to two inches from the perimeter, the timber seemed “spongy”.

Immediately beneath the timber deck, the soil was extremely soft with essentially no support capacity.

Within the approximately 6-foot wide cavity opening, the exposed seaward row of “support piles” apparently did not provide support to the base of the cast-in-place concrete seawall either by direct bearing or via a pile supported timber platform.

Landside Investigation

The landside inspection again revealed that the seawall was constructed generally consistent with the archive drawings. The measured step down pattern on the landside closely correlated with the step down pattern indicated in the archive drawings. The deck timbers appeared well preserved. However, the wood fibers were very saturated with water and seemed "spongy". The spongy fibers extended approximately an inch inward from the faces of the timbers before solid resistance could be felt. Deep grooves could easily be impressed into the wood decking by a screwdriver shaft with very little effort.

The top portion of a single vertical support pile was unearthed. Inconsistent with the archival sketch on page 1 of Appendix F, the center of the pile was approximately 2'-6" seaward from the landward edge of the deck. The archival sketch indicates that the center of the pile was to be located approximately 6 inches from the landward edge of the deck.

A portion of the deck was removed during the investigation to better observe the condition of the platform framing system. A pair of 5" wide x 12" deep timbers was bolted into the sides of the pile.

Here also, the bolts were very severely corroded from the ends with significant loss of metal cross section. A screwdriver shaft could be inserted into the ends of the bolts and pushed several inches into the interior length of the bolts. Again, the remaining cross-section of a bolt was more comparable to a thin walled pipe than to a solid round rod.

The investigation revealed that approximately two inch wide by twelve inch deep notches had been cut into the sides of the piles to provide partial end support for the 5" wide x 12" deep timbers

Station 27+48
(in the vicinity of 32 Murray Boulevard)

This site location corresponds to the right side of the top photograph on page 24 of Appendix B. Signs of deterioration include rust streaks from exposed and corroding steel reinforcing bars in the coping and horizontal cracks below the coping.

Seaside Investigation

In order to investigate, an original cementitious patch covering the head of one of the steel bolts connecting the concrete veil to the underlying timber sheet pile and timber waler was removed. The exposed head of a steel bolt was found to be in very good condition and had very little corrosion. A core sample of the protective concrete veil and a core sample of the underlying timber sheet pile structure were taken. Both the concrete and the wood were in very good condition. The concrete quality was very good, and the joint between the bottom of the seawall and the protective concrete veil appeared sound.

Landside Investigation

The landside investigation uncovered two support piles located approximately six feet apart. The associated 5” x 12” timber beams were connected to the sides of the piles near their tops. The construction was consistent with the archive drawings.

The platform deck timbers, the 5” x 12” timber beams, and the top portions of the support piles appeared well preserved. However, the wood fibers were very saturated with water and seemed “spongy”. The spongy fibers extended approximately an inch inward from the faces of the timbers before solid resistance could be felt. Again, deep grooves could easily be impressed into the wood decking by a screwdriver shaft with very little effort.

The bolts, washers, and nuts used in the connection of the 5” x 12” timbers to the piles again were severely corroded. A screwdriver shaft was pushed in its full 12 inches into the interior length of one of the bolts and six inches into the interior length of another bolt before any resistance could be felt.

Station 43+50
(near White Point Garden between the King Street and the extension of Meeting Street)

This station is included in “The Boulevard Extension” portion of the Low Battery seawall. “The Boulevard Extension” extends between the eastern end of White Point Garden and King Street and was completed approximately ten years after the “Boulevard” section of the Low Battery seawall. “The Boulevard Extension” also appears to have been constructed to the dimensions and configuration presented in the archival design sketch on page 1 of Appendix F.

This station corresponds to the photographs on page 17 in Appendix B. The prominent sign of deterioration on the seaward side is the wide horizontal crack located approximately six feet below the coping. The crack appears similar to the crack at Station 9+73 near 84 Murray Boulevard.

This type of crack repeats along much of the seaward length of this seawall section. In the past, attempts have been made to seal the wider portions of the crack with patches of bricks and mortar.

Seaside Investigation

The investigation was performed at the existing crack opening in the face of the seawall. At the selected station, the crack opening was approximately 6 inches wide with corroded reinforcing bars exposed to view.

The concrete was of very poor quality. The concrete was composed predominately of coarse aggregate with very little fine aggregate or cement included to provide bond. Again, it was relatively easy to excavate through the face of the seawall by removing existing broken sections of concrete with pneumatic hand tools. The resulting cavity opening in the face of the seawall was approximately 2 feet high by 3 feet wide. The cavity extended approximately 2 feet into the seawall.

Two batter piles in the seaward row of piles were exposed to view by this opening. No vertical support piles or the accompanying 5" x 12" timber beams were exposed by this relatively small opening. In contrast to the findings at Station 9+73, the tops of both of the batter piles were in firm contact with the underside of the cast-in-place concrete seawall and the timber support platform extended completely to the seaward face of the seawall.

Where exposed to view, the perimeters of the top portion of the batter piles were soft and the exterior one inch of wood fiber was easily scrapped away. There was no apparent marine borer damage to the two batter piles. The timber platform deck, located just below the surface of the mud, had no apparent marine borer damage.

In this location, the timber sheet pile system extended approximately one foot above the timber platform and the protective concrete veil. The portions of the timber sheet pile system extending above the mud line were actively infested with marine boring organisms.

Those portions of the steel reinforcing bars extending from the ends of the concrete veil into the base of the seawall were exposed to the weather and were severely corroded. However, the top edges of the exposed portion of the concrete veil sections appeared intact and in good condition.

Landside Investigation

The landside excavation exposed the timber support platform and the top portion of a vertical support pile connected to a pair of 5" wide x 12" deep timber beams.

The underside of the timber platform had considerable marine borer damage, although no signs of active marine borers were present. It is possible that the marine borer damage occurred during the original construction phase when the platform extended over then-open water. The subsequent placement of the earth fill on the landside of the seawall suffocated the marine borers. It is noted that the topside of the timber platform did not have any marine borer damage.

Again, the 5" x 12" timber beams and the top portion of the pile were saturated with water and seemed "spongy". The spongy fibers extended approximately an inch inward from the faces of the timbers before solid resistance could be felt. Here again, deep grooves could easily be impressed into the wood decking by a screwdriver shaft with very little effort.

Similar to the other landside investigations, the steel bolts, nuts, and washers used in the connection of the 5” x 12” timber framing to the support piles were severely corroded. The ends of the bolted connections were heavily encrusted with a ferrous compound. The interior of the lengths of the bolts had corroded deeply inward from the ends. Here also, a shaft of the screw driver could be inserted into the ends of the bolts and pushed several inches into the interior length of the bolt. Also at this station, the ends of the 5” wide timbers are supported by the 2 inch wide notches cut into the sides of the support piles.

Concrete Extension of the High Battery Seawall

As-built drawings indicating the cross section design through the concrete extension of the High Battery Seawall could not be located during the research phase. However, the seaward portion of the seawall appears to be constructed utilizing the same basic design concept of the Low Battery seawall, yet, elevating the entire structure to achieve the desired higher deck elevation. The timber sheet piles and the protective concrete veil extend approximately three feet above the mud line. Six inch thick concrete walkway deck slabs are supported by and span between the top of the seaward seawall and the concrete landside wall. Beneath the concrete walkway deck slabs, a series of interior concrete beams extend radially seaward from the curve of the landside wall to the curve of the seaward seawall. These beams provide primary vertical support for the concrete deck slabs and horizontal bracing for the walls. Soil borings taken through the deck slabs in 1997 by Soil Consultants, Inc. indicated that the chambers formed between the interior beams contain varying depths of earth fill.

Also beneath the concrete walkway deck slabs, a series of concrete counterfort walls are perpendicular to the seaward seawall on its landside. The counterfort walls provide bracing for the seaward seawall.

Photographs, field notes, and sketches from the detailed investigations are included in Appendix N.

Station 49+24

The seaward location corresponds to the top photograph on page 21 of Appendix A. The most obvious sign of deterioration on the seaward side is the large hole penetrating the protective concrete veil. The hole is located approximately two feet above the mud line and almost directly below the bronze plaque on the walkway. Similar holes of varying sizes are seen elsewhere along this section of the seawall.

Seaside Investigation

The existing hole in the protective concrete veil was roughly circular in shape and approximately one foot in diameter. Seawater was visible in a large cavity behind the concrete veil. Seawater flowed

in and out with the tide.

Severely deteriorated sections of concrete and corroded reinforcing bars were removed from around the existing hole in the concrete veil. The resulting opening, approximately 1 foot high and 3 feet wide, provided access to the large cavity behind the concrete veil. The floor of the cavity resembled a gently sloping sand, pebble, and rubble beach. From the opening in the face of the seawall, the floor or beach of the cavity extended landward with a gentle upward slope for approximately 8 ½ feet. At this interior location, the floor and ceiling contacted. The ceiling of the cavity is the underside of the cast-in-place portion of the seawall. Additional probing at the back of the cavity under the cast-in-place portion of the seawall encountered solid resistance approximately one foot further landward.

Inside the opening in the seawall face, the cavity narrowed in width and decreased in height as it extended to the north and to the south. A metal tape measure was extended approximately 8 feet to north and 12 feet to the south of the opening along the interior face of the concrete veil before encountering resistance.

Inside the opening, four severely deformed, cone-shaped remnants of the original support piles projected approximately twelve inches above the mud line. These piles were part of the seaward row of support piles. The tops of the piles were not in contact with the concrete seawall. The exposed underside of the concrete seawall was approximately six inches above the tops of these piles. The pockets that had been formed during the original construction phase by the concrete flowing around the tops of the support piles and the timber sheet piles were visible.

Without the necessary pile support, the timber beams and timber platform have long since collapsed. Currently, only the severely deteriorated remnants of the 5” wide x 12” deep timber support beams were visible at the mud line. There were few visible remnants of either the timber support platform or the steel connection bolts.

At this station, with long term exposure to seawater, the visible portions of the structural support system of the seaward seawall apparently has been completely destroyed by the marine boring organisms.

Station 49+21

Landside Investigation

This station corresponds to the photographs on page 9 of Appendix A. The investigation revealed the landside seawall to be a concrete wall approximately nine feet in overall height from the underside of the concrete deck walkway to the wall’s base below ground. The concrete wall was approximately one foot thick.

The construction of the landside seawall had many of the familiar construction patterns uncovered in the other investigated landward sites. A timber platform supported the base of the concrete wall and extended approximately one foot landward from the base of the wall. The investigation uncovered a pair of 5" wide x 12" deep timber beams bolted to the sides of a vertical timber support pile near its top. Two inch wide seats were notched into the sides of the piles to provide partial end support for the beams. The bolts, washers, and nuts used in the connection had the typical severe corrosion.

Again, all of the timbers of the support structure were saturated with water and seemed "spongy".

Observations through a cored hole in the 6" thick deck slab confirmed the existence of a void chamber under the concrete walkway slab. The seaward seawall appeared to have the familiar step down pattern on its landward side. However, a series of concrete counterfort walls were perpendicular to the seaward seawall on its landside. Furthermore, the underside of the concrete deck appeared to be severely deteriorated.

Marina Seawall

As-built drawings of the Marina seawall could not be located during the historic research phase. The detailed investigation revealed that this seawall is constructed of tabby concrete and is supported on a timber mat foundation constructed with logs. Photographs, field notes, and sketches from the detailed investigation are included in Appendix O.

Station 2+38

The conditions at this chosen site location correspond with the typical conditions displayed in all of the photographs in Appendix C. The site is along the eastern leg of the seawall near its southern corner.

Seaside Investigation

The top of the tabby concrete seawall is approximately 4 ½ feet above the mud line. The seawall is approximately 2 feet wide at the top. The top surface of the tabby concrete seawall is covered with a thin topping of Portland cement concrete. Along portions of the wall, the seaward face is severely deteriorated above the mud line. Deep, wide crevices located above the mud line extend horizontally along much of the seaward face of the seawall.

Near the observation station, a deep vertical crack opens to a large cavity centered within the seawall. Cylindrical in shape, the cavity is approximately 12 inches in diameter and extends vertically approximately three feet above the mud line. Similar vertical cracks and cavities occur at approximately ten foot intervals along the length of the seawall. The cavity dimensions and spacing

suggest that the seawall was originally constructed with tops of timber piles projecting up into these cavities. With the tops of the piles anchored into the seawall and the bottom portion of the piles anchored in the mud, the seawall gained additional capacity to resist overturning moments and horizontal forces.

A horizontal row of cavities with a rectangular cross-section approximately 13 inches wide by 14 inches high extends between the centerline of the vertical cavities. It is believed that these horizontal cavities originally contained timber structural members linking the piles together in the longitudinal direction.

Cracks and crevices in the seaward face of the tabby seawall permitted the entrance of seawater and exposure to marine boring organisms. Over the years, decay and marine boring organisms have destroyed the internal timber members located above the mud line.

Interestingly, the excavation below the mud line adjacent to the seawall revealed an extensive timber mat constructed with palmetto logs, at least four layers deep. The original intent of the timber mat is unknown. A consideration is that the timber mat served as a temporary road during the original construction of the seawall. All the layers of palmetto logs extend parallel with the face of the seawall. The top surface of the top layer of logs was approximately one foot below the mud line. The palmetto log mat continued seaward and along the length of the seawall beyond the limits of our excavation pit.

The base of the tabby seawall was approximately 6’-8” below the top of the seawall and approximately level with the bottom of the forth layer of these palmetto logs. A foundation mat composed of at least one layer of palmetto logs extending perpendicular to the seawall supported the base of the tabby concrete seawall. The logs appeared well preserved.

Landside investigation

Close proximity to an asphalt parking lot and the in-situ rubble backfill severely restricted the size and depth of the observation pit. At this station, a vertical crack approximately 2 inches in width penetrated through the seawall to its seaward face. The crack has permitted the fines in the soil backfill to be displaced by tidal action. The erosion of the backfill adjacent to the landside face of the seawall is extensive.

Approximately four feet below the top of the seawall, the remains of the original formwork used in the construction of the tabby concrete were unearthed. The formwork was constructed using cypress boards 8 inches wide by 1 ½ inches thick. The rubble backfill prevented deeper excavation. Using a probe, a timber layer, believed to be the timber mat foundation layer, was detected approximately 8 feet below the top of the seawall.

PHASE III GENERAL CONCLUSIONS

Stone Masonry Portion of the High Battery Seawall

At the investigated station, the landside wall of the stone masonry portion of the High Battery Seawall appears to have been constructed closely corresponding to the cross section sketch on page 1 of Appendix E.

The Low Battery Seawall

The Low Battery seawall generally appears to have been constructed to the dimensions and configuration set forth in the archival design sketch on page 1 of Appendix F.

At two of the investigated stations, the most obvious signs of structural deterioration on the seaward side of the seawall are the wide, horizontal cracks located approximately six feet below the coping. Each of the cracks is approximately 95 feet long and extends between vertical expansion joints in the seawall face.

The investigation revealed that the prominent horizontal cracks were not occurring along the joint between the base of the pile supported seawall and the array of protective concrete slabs. Rather, the long horizontal cracks were actually occurring several feet above this joint within the concrete mass of the pile supported seawall.

The investigation of two of the stations where the cracks occurred revealed that poor quality concrete exists in the seawall face in the zone below the crack and above the mud line. The concrete is soft and easily removed with pneumatic hand tools. At the two stations, the investigation revealed that the level of deterioration caused by marine boring organisms to the seaward timber support structure appeared to be fairly minimal.

For the portions of the timber support structures projecting above the mud line, it appeared that their only protection in fending off destruction by marine boring organisms was a fortuitous thick coating of mud adhering to their perimeters.

Nevertheless at these two investigated sites, seawater and the accompanying marine boring organisms have made a deep access into the seawall structure through the crack openings in the face of the seawall. The portions of the timber support structure above the mud line are at continuing risk of attack by marine boring organisms.

The landside and seaside investigations revealed that the bolts, washers, and nuts used in the connection of the 5” x 12”support timbers to the piles were severely corroded. It is very apparent

that the steel bolts have little remaining capacity. The notched seats in the piles essentially provide all of the vertical end support for the beams. At best, the remnants of the bolted joint provide some nominal amount of rotational resistance for the connection. Furthermore, the timbers of the support structures were saturated with water and seemed "spongy".

Concrete Extension of the High Battery Seawall

The seaward portion of the seawall appears to be constructed utilizing the same basic design concept of the Low Battery seawall yet elevating the entire structure to achieve the desired higher deck elevation.

The protective concrete veil has been repeatedly and extensively breached. At the investigated station, marine boring organisms have caused nearly total disintegration of the visible timber support structure of the seaward seawall. Also the earth fill has completely washed out from between the seaward seawall and the landside wall. The seawall's ability to resist substantial vertical loads and environmental loads such as hurricane force winds and the accompanying wind driven waves is seriously compromised.

At this station, the end of the concrete deck slab is badly deteriorated at the landside seawall support. Furthermore, the undersides of the concrete walkway deck slabs are significantly deteriorated. "As-built" drawings indicating the design of the concrete deck slabs could not be located during the research phase. Without additional testing, it may not be possible to accurately quantify the remaining structural capacity of the concrete walkway deck slabs. Nevertheless, the slab's original structural capacity is seriously compromised. The concern is that under certain conditions of heavy pedestrian loadings, a walkway deck slab could become overstressed and give way.

Marina Seawall

At this investigated station, the portion of the tabby concrete seawall just above the mud line is in a very severely deteriorated condition. The top portions of timber piles and other horizontal timber members that were originally internal to the tabby concrete have been significantly reduced in cross-section and/or completely eliminated by marine boring organisms. The seawall's ability to resist overturning earth pressures and lateral load due to high wind and wave action is seriously compromised. Nevertheless, the seawall remains in place.

With Phase III of the seawall study complete, the construction materials, the geometry, and the conditions of the seawalls are now better known. The next phase of the study is the comprehensive engineering analysis and proposed repair recommendations. The goals of the analysis include determining conceptual repair and/or stabilization options; estimating construction costs, and establishing priorities for repairs and stabilization of the seawalls.

PHASE IV – THE COMPREHENSIVE ENGINEERING ANALYSIS AND REPAIR RECOMMENDATION PHASE

SCOPE OF WORK

The goals of the comprehensive engineering analysis and proposed repair recommendation phase include determining conceptual repair and/or stabilization options, estimating construction costs, and establishing priorities for repairs and stabilization of the seawalls.

TERMINOLOGY

A seawall is a soil retaining and an armoring structure whose purpose is to defend the shoreline against wave and water erosion.

The life span of a seawall is difficult to accurately predict or even to clearly define. The definitions for the "lives" used in this report are based on those outlined in the reference manual Economic Analysis Handbook, NAVFAC P-442, 1975.

Some of a seawall's defined lives include service life, physical life, economic life, mission life, and technological life. The *service life* is defined as the period of time during which the seawall performs the function for which it is designed and used. The *physical life* is defined as the period over which the seawall may be expected to last physically. *Economic life* is the period of time during which the seawall provides a positive benefit. The *mission life* is the period over which a need for the seawall is anticipated. *Technological life* is the period before which obsolescence would dictate replacement of the existing seawall.

It is reasonable to conclude from the above definitions that a seawall has different life spans depending on the role it is capable of performing or the purpose for the seawall.

The seawalls of Charleston must be able to successfully withstand major hurricanes without significant damage. This need is both historic and ongoing.

SERVICE LIFE

The service life of a seawall is finite. Any structure in the marine environment is in an ongoing state of slow deterioration. The overall long-term durability of a given structure depends upon local environmental conditions, design practices, the materials of construction, quality of construction, and maintenance practices. The service life span of a seawall is affected by all these factors. However,

when considering an already existing seawall, most of these factors are beyond our control. Though the local environmental conditions are a given, the original design practices, the materials of original construction, and the quality of original construction are unalterable details of an already existing seawall structure. As a result, only maintenance practices remain available for consideration in extending the service life of such existing seawalls.

EXTENSION OF SERVICE LIFE

The potential for extending the service life of a structure depends primarily on the structure's maintainability. From the reference manual, Durability of Engineering Structures by Jan Bijen, 2003, maintainability is defined as "the extent to which it is feasible that the performance of a structure will be restored to the intended original level within a given period of time. The two factors that determine maintainability are accessibility of the structure to inspection so as to allow for proper diagnosis and ease of remedial action to restore structural performance."

A very significant portion of a seawall, the subterranean foundation, inherently lacks maintainability by the nature of the design. It is not readily accessible. Nor would it be easy to restore the original structural performance if required. Even for the exposed seaward face of a seawall, maintenance to restore the original structural performance is difficult and expensive within the tidal zones.

CATEGORIES FOR MAINTENANCE

In general, maintenance refers to all activities aimed at maintaining and/or restoring a specified performance of an existing structure.

The categories of maintenance used in this report are based on the guidelines outlined in the reference manual, Durability of Engineering Structures by Jan Bijen, 2003. The three categories for maintenance of the seawalls are prescribed preventive maintenance, reactive preventive maintenance, and corrective maintenance. They are defined as follows:

1. Prescribed preventive maintenance includes tasks carried out periodically, even before there are any apparent signs of moderate deterioration,

2. Reactive preventive maintenance includes tasks taken when deterioration is moderate, but the overall structural capacity has still not been severely compromised,

3. Corrective maintenance includes the necessary repairs to restore the ability of the structure to function at the capacity desired when the deterioration is major and the structural capacity has been severely compromised.

REPLACEMENT ALTERNATIVE

It would seem that if enough money were spent on maintenance of an existing seawall, then the service life of the structure could be extended indefinitely. However, this "last-for-ever" approach of maintenance on an existing seawall structure may not be practical or economical.

When maintenance is no longer considered economical or practical, replacing an existing structure with a new structure and providing a succession of service life remains the only option.

Interestingly, the replacement alternative has been utilized several times for the stone masonry portion of the High Battery seawall. The gale of 1854, the hurricane of 1885, and the hurricane of 1893 significantly damaged the seawall.

With the passing of each of these storms, the previous design of the seawall was replaced with a stronger design. At present, a dual seawall system is in place complete with interior concrete backup walls and earth fill in between. Through an evolutionary process, the original design of the stone masonry portion of the High Battery seawall has essentially been replaced three times and each replacement alternative has incorporated the original stone masonry portion of the seawall.

It is clearly not necessary for a replacement alternative to disregard the earlier structure.

APPROACHES TO MAINTENANCE OR REPLACEMENT

The desired goals for the stone masonry portion of the High Battery seawall, the Low Battery seawall, the concrete extension to the High Battery seawall, and the Marina seawall and the historic significance of each individual seawall affect the approaches to maintenance or replacement thereof.

For this engineering study, two different approaches to maintenance or replacement efforts are being considered based on different desired goals. The two approaches will be classified as preservative and fundamental.

The preservative approach to maintenance or replacement will be that effort necessary to preserve and safeguard the historic outward character of the seawall structure as well as providing for the extension or succession of the structure's service life.

The intent of the preservative approach to maintenance of seawalls parallels the stated intent of the City of Charleston's Board of Architectural Review with regard to the maintenance and preservation of historic structures. The stated intent of the Board of Architectural Review includes "the preservation and protection of the old historic or architecturally worthy structures and quaint neighborhoods which impart a distinct aspect to the City of Charleston, the state, and the nation. …In

an effort to retain this texture, wholesale replacement of materials is discouraged. Elements should be repaired, rather than replaced wherever possible. ”

The major goals to develop the preservative approach to seawall maintenance or replacement are as follows:

1. Successfully withstand major hurricanes without significant damage,
2. Successfully serve as a soil retaining structure,
3. Preserve and maintain the historic outward character of the seawall.

The fundamental approach to maintenance or replacement will be considered as that effort necessary to provide for the extension or succession of the structure’s service life without the extraordinary effort to preserve material pieces of the original construction. The fundamental approach will make use of modern day materials, design practices, and construction practices. Nevertheless, the exposed portions of the seawalls shall continue to match the historic profile and the material texture of the original construction.

The major goals to develop the fundamental approach to seawall maintenance or replacement are as follows:

1. Successfully withstand major hurricanes without significant damage,
2. Successfully serve as a soil retaining structure,
3. Match the historic profile and the original material texture of the exposed portion of the seawall.

SUGGESTED APPROACHES

The stone masonry portion of the High Battery seawall with its historic significance, character, and location warrants the preservative approach.

Although considerably less renown, the tabby concrete Marina seawall is a remarkable historic civil engineering structure and also deserves the preservative approach.

The concrete extension of the High Battery seawall and the Low Battery seawall are clearly important seawall structures but with significantly less historic character. The exposed seaward faces of these structures are of nondescript comparatively modern concrete. Maintaining the historic profile and the material texture of the concrete seawalls seems reasonable, but an extraordinary effort to preserve pieces of the older yet modern concrete, seems unwarranted. For the concrete extension of the High Battery seawall and the Low Battery seawall, the fundamental approach seems the most reasonable.

Ultimately, it will be the City of Charleston as the owner of the seawalls to conclude their desired approaches to maintenance or replacement.

PRACTICALITY OF OPTIONS

The basic conservative intent of both the preservative and the fundamental approach restricts the options available for maintenance or replacement.

For the Marina seawall as one example, the concept of covering the existing seawall structure for its entire length and height with stone riprap would not meet the intent of preserving and safeguarding the historic outward appearance. Likewise, the concept of constructing a new seawall to seaward of the existing structure and backfilling between the structures with earth fill, would not meet the intent of preserving and safeguarding the historic outward appearance. Similarly, the concept of merely patching the holes in the tabby concrete seawall, which does not substantially increase the structure's strength, would not meet the intent of being able to successfully withstand major hurricanes without significant damage.

Concepts for maintenance or replacement need to meet all of the intended goals.

ESTIMATED COSTS OF MAINTENANCE OR REPLACEMENT

Estimates of construction costs are made based on fiscal year 2004 dollars. The costs of construction were estimated utilizing the reference manual Heavy Construction Cost Data 2004, by R.S. Means Company, Inc. Added insight into estimated costs was obtained from conversations with various local specialty and marine contractors who have performed work similar to that envisioned.

MAINTENANCE PRIORITIES

It should be recognized that establishing and ranking the maintenance priorities among the historic seawalls is subjective in nature. Two individuals presented with the same information could reach different conclusions based on their respective viewpoints. In this report, we feel that the establishing and ranking of the priorities has been made from a conservative viewpoint of the priorities.

The sequence of factors that influence the recommended ranking of seawall maintenance priorities is as follows:

1. The importance to peninsular Charleston for the seawall to successfully withstand major hurricanes without significant damage,
2. The seawall's physical location and extent of exposure if impacted by major hurricanes,
3. The severity of existing deterioration to the seawall,

4. The estimated ongoing rate of deterioration to the seawall,
5. The visibility of the seawall's physical location,
6. The potential impact if corrective maintenance is deferred,
7. The potential impact if preventive maintenance and reactive preventive maintenance are deferred.

SEQUENCE OF PRIORITY AMONG THE SEAWALLS

The recommended sequence of priority among the historic seawalls for maintenance is as follows:

1. The Concrete Extension of the High Battery Seawall,
2. The Low Battery Seawall,
3. The Stone Masonry Portion of the High Battery Seawall,
4. The Marina Seawall.

CAVEAT

As previously stated in this report, localized site investigations of the various seawalls were performed. They provided insight to the condition defects probable elsewhere in the seawall.

Realistically, these site investigations can not provide precise values of either the quantity or the extent of the condition defects that actually exist within the very substantial unexposed portions of the seawalls.

CHARACTER IMPROVEMENTS

The effort to provide the necessary maintenance or replacement for the seawalls can fuel the desire to simultaneously make character improvements to the seawalls. For instance, replacing the existing safety railing system on the stone masonry portion of the High Battery seawall with a replica of the more historic iron railing system would be an example of a character improvement.

It is exciting to imagine all the possibilities for character improvements that could be made to the various seawalls. The possibilities are nearly without limit in both scope and construction costs. Making desired character feature improvements to the seawalls simultaneous with ongoing necessary maintenance repairs will always be an option to consider for the City of Charleston.

Nevertheless for this report, character improvements will be considered outside of the basic intent of providing maintenance to an existing seawall.

ANALYSES AND RECOMMENDATIONS

The Concrete Extension of the High Battery Seawall

EXISTING CONDITION

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the seaward seawall and the concrete walkway are in poor condition. Previous sections of this report document the observed details of their physical condition.

Marine boring organisms have caused near total disintegration of the visible timber support structure of the seaward seawall. Also, the earth fill has completely washed out of one of the zones between the seaward seawall and the landside seawall. With such extensive deterioration of critical support structure, it is remarkable that there are not more visible indications of resulting distress and instability.

The undersides of the concrete walkway deck slabs are significantly deteriorated. Areas on the topsides of the concrete walkway have extensive cracks and major spalls. Furthermore, the concrete walkway is partially supported by the seaward seawall with its severely deteriorated foundation.

It is believed that the curved shape of this seawall, the interior counterfort walls, the concrete deck beams supporting the concrete walkway, and to a lesser extent the landside seawall all contribute to providing an unintended redundant "back-up" support system for the seaward seawall. It is virtually impossible to predict how much additional deterioration the seaward seawall can withstand or the remaining capacity of the unintended support system.

POTENTIAL LIABILITY

The seaward seawall portion and the concrete walkway portion of the structure appear to be at or near the end of their respective service lives. The seaward seawall portion and the concrete walkway portion either need to receive extensive corrective maintenance or be replaced by new structures.

The seawall's ability to resist substantial vertical loads and environmental loads such as hurricane force winds and the accompanying wind driven waves is seriously compromised. In its present deteriorated condition, it seems doubtful that the concrete extension of the High Battery seawall could successfully withstand the direct onslaught of a major hurricane without substantial damage. The concrete walkway slab's original structural capacity is severely compromised.

The seawall is situated at the relatively exposed southeast tip of the peninsula and essentially faces towards the open mouth of Charleston Harbor. When propagated from a southeast direction, wave action impacts directly upon the concrete extension of the High Battery. With considerable upwind fetch over open water in the southeast direction, the energy levels of hurricane force wind driven waves would be very substantial.

The seawall protects the intersection of Murray Boulevard and East Battery. It protects White Point Garden beyond. There are no homes immediately adjacent on the landside of the seawall. Nevertheless, if this seawall should be significantly breached during a major hurricane, hurricane driven waves could propel flood waters well into the southeastern portion of the peninsula.

CORRECTIVE MAINTENANCE OR REPLACEMENT

Initially, corrective maintenance was considered for repairing the deteriorated support structure of the existing seaward seawall.

The sequence for the maintenance concept considered was as follows:

1. Obtain necessary permits from Federal and State Regulatory Agencies,
2. Construct a cofferdam around the perimeter of the seaward seawall,
3. Excavate under the existing concrete seaward seawall and remove the deteriorated portions of the support piles,
4. Excavate further below the mud along the lengths of the existing piles,
5. Splice on concrete pile jacket extensions and necessarily achieve full moment connections between the timber piles and their concrete pile jacket extension,
6. Achieve a substantial connection at the top of the concrete pile jacket extension with the underside of the existing concrete seaward seawall,
7. Replace the seaward concrete veil, etc…

Conducting extensive construction operations under the seaward seawall with its overhanging concrete mass of questionable stability would be a great safety concern to all involved. Furthermore, the weight of the concrete mass would not automatically shift back onto the timber piles via the extensions. A subsequent and potentially undesirable movement, such as a long term deflection or tilting of the concrete mass, would be necessary to reintroduce even a portion of the original weight back onto the timber piles via the extensions.

Attempting to return the seaward seawall to its original condition and strength capacity by corrective maintenance seems elusive at best. In general, the corrective maintenance construction process necessary to preserve the original concrete mass of the seaward seawall seems costly and of questionable practicality.

In consideration of the extent, nature, and location of the deterioration to the timber pile support structure and timber support platform, it appears that for the seaward seawall portion, the replacement alternative would be both the more economic and practical choice. Furthermore, since the timber foundation system of the seaward seawall also provides apparent stability to the foundation of the landside seawall, it would be more economical and practical to remove and replace the landside concrete seawall as well. Also, the concrete walkway deck slabs should be removed and replaced.

PROPOSED REPLACEMENT CONCEPT

For the concrete extension to the High Battery seawall, the proposed concept envisioned would utilize the fundamental approach to replacement. A sketch of the proposed replacement concept is included in Appendix P.

The sequence for the proposed concept envisioned would be as follows:

1. Obtain necessary permits from Federal and State Regulatory Agencies,
2. Construct a temporary cofferdam around the seaward perimeter of the seaward seawall,
3. Construct a temporary sheet pile retaining wall system around the landside perimeter of the landside seawall,
4. Remove the existing concrete walkway panels,
5. Excavate and remove the existing earth fill remaining between the seaward and landside seawalls,
6. Excavate and remove the existing earth between the landside seawall and the temporary sheet pile retaining wall system as necessary,
7. Demolish and remove the concrete deck beams,
8. Demolish and remove the concrete seaward seawall, interior counterfort walls, and any projecting concrete veil structure,
9. Demolish and remove the concrete landside seawall,
10. After testing, reuse the preserved length of the existing piles continuously buried in the mud if the piles still have substantial load carrying capacity,
11. Drive new supplemental prestressed concrete piles as necessary,
12. Reinforce the existing earth subgrade to provide temporary support until the new concrete could harden,
13. Construct a new concrete seaward seawall with its base extending several feet below the mud line,
14. Construct a new concrete landside seawall,
15. Replace the concrete deck beams,
16. Place new earth fill between the seawalls and install new concrete walkway panels,
17. Install new railing system and replace stairs,
18. Place new earth fill between the new landside seawall and the temporary sheet pile retaining wall system and remove the temporary sheet pile retaining wall system,

19. Place additional stone riprap around the seaward perimeter of the new seawall and remove the temporary cofferdam system.

Necessary permits would have to be obtained from Federal and State Regulatory Agencies prior to commencing construction on the seaward side.

The concept would entail installing a temporary cofferdam system seaward of the perimeter of the concrete extension of the High Battery seawall. Construction operations would need to be carried out regardless of the state of the tide.

The concept would also entail installing a temporary sheet pile retaining wall system around the landside perimeter of the landside seawall. The temporary retaining wall system would facilitate subsequent excavations adjacent to the street.

The existing concrete walkway panels would be removed. The existing earth fill remaining between the seaward and landside walls would be excavated and removed from the site.

The concrete deck beams would be cut away from the interior of the landside seawall and the interior of the seaward seawall and removed.

The concrete seaward seawall, the interior counterfort walls, any projecting concrete veil structure, and the landside seawall would then be demolished and removed from the site. The temporary cofferdam structure would prevent seawater from intruding.

Unlike the existing seaward seawall with its base situated several feet above the mud line, the base of the proposed new seaward seawall would extend several feet below the mud line. Consequently, the potential for scour under the seawall structure would also be reduced.

The proposed new design would allow effective reuse of the preserved portion of the existing piles; that portion continuously buried in the mud. The deteriorated tops of the existing timber piles would be cut off. After testing, the preserved length of the existing piles continuously buried in the mud would be reused if the piles still have substantial load carrying capacity.

Nevertheless, the proposed new design of the deeper seaward seawall would increase the weight on the piled foundation. In order not to increase the potential load on any of the existing timber piles, new supplemental prestressed concrete piles as necessary would be driven.

Any soft earth around the top several feet of the piles would be removed. In its place, crushed stone would be installed and pressed into the earth subgrade to strengthen it. The stone reinforced subgrade would provide a firm temporary support around the pile heads until the freshly placed concrete could harden.

The seaward seawall would be constructed in several vertical stages or lifts as necessary and the

profile of the seaward face would match the profile of the existing seawall. Likewise the exposed profile of the landside seawall would be constructed to match the exposed profile of the existing landside seawall.

The new replacement concrete deck beams would subsequently be constructed.

Subsequently, new earth fill would be placed between the seawalls and new concrete walkway panels installed. Earth fill would be placed between the new landside seawall and the sheet pile retaining wall system as necessary to restore the original ground elevations and subsequently the temporary sheet pile retaining wall system would be removed. Additional stone riprap would be placed around the seaward perimeter of the new seawall and the temporary cofferdam system removed.

CONCEPTUAL ESTIMATE OF COSTS

The conceptual estimate of total engineering design and construction costs is $ 1,800,000. Additional details of the estimated costs are included in Appendix Q.

The Low Battery Seawall

EXISTING CONDITION

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the exposed portions of the seaward face of the concrete seawall range from poor condition to fair condition. Previous sections of this report document the observed details of the physical condition.

As previously stated in this report, the most obvious signs of structural deterioration on the seaward side of the seawall are the wide, horizontal cracks located approximately six feet below the coping. Each of the cracks is approximately 95 feet long and extends between vertical “expansion” joints in the seawall face. These cracks were occurring within the concrete mass of the pile supported seawall.

Poor quality concrete exists in the seawall face in the zone below the cracks and above the mud line. With the deep crack openings in the face of the seawall structure, seawater and the accompanying marine boring organisms have apparent access deep into the seawall structure. The portions of the timber support structure above the mud line are at continuing risk of attack by marine boring organisms.

Also previously stated in this report, both the landside and seaward side investigation revealed that the timbers of the support structure were saturated with water and seemed “spongy”, that the bolts,

washers, and nuts used in the connections of the timber support structure were severely corroded, and that the notches in the piles essentially provide all the vertical end support for the 5” x 12” beams.

It is believed that the timber support platform was originally constructed as a platform over then open water to expedite construction and to support the base of the newly constructed concrete structure. Once the backfill was placed and the overall structure became a seawall/retaining wall, the purpose of the timber platform became less critical. The timber platform and the landside row of vertical piles add to the overall stability of the seawall structure, however the seaward rows of vertical and batter piles provide primary direct foundation support for the concrete seawall structure.

Currently, the steel connection bolts are severely deteriorated. The connections of the 5” x 12” timber beams to the tops of the timber piles predominately rely on a combination of support/restraint factors including bearing on the notches in the pile tops, friction, and earth pressures. With no visible signs of settlement, the foundation support system appears to be functioning satisfactorily.

The vast quantity of stone riprap that has been placed subsequent to construction along the entire length of the seaward base of the seawall is a curiosity. From the historic research phase, it was learned that some stone rip rap was placed in discreet areas to reinforce the timber sheet pile system where a localized blowout had occurred during the placement of the earth backfill. However, the vast quantity of riprap presently at the base of the seawall has little correlation with the amounts necessary for the above mentioned localized repair.

Nevertheless, the extensive rip rap provides stability. The rip rap has essentially been placed to the top of the sheet pile system. The severely deteriorated steel bolts connecting the timber sheet pile system to the timber walers have little remaining strength. The stone rip rap reduces the importance of the deteriorated steel bolted connection. The stone rip rap provides passive earth pressure restraint and an anchoring effect to the top of the seaward face of the sheet pile system. With no apparent signs of outward movement at the top of the sheet pile system at the present time, it appears that the stone rip rap is functioning satisfactorily in providing the necessary stabilization.

TILTING SIDEWALKS

The concrete sidewalks along the landside of the seawall are conventional slab-on-grade sidewalks. The tilting of the sidewalk is irregular and appears to be primarily the result of long-term settlement of the earth fill placed on the landside of the seawall. There did appear to be some localized erosion of earth subgrade immediately adjacent to the occasional storm drain outlet through the seawall.

All along the seawall, the seaward end of the sidewalk is supported by earth fill which is supported by the step-down portion of the pile-supported seawall and the pile-supported timber platform.

In essence, the seaward side of the sidewalk is indirectly supported by the pile-supported seawall/platform that has not settled and the landside end of the sidewalk is directly supported by

earth fill that has been settling over the years.

There was no indication of a seaward movement of the earth subgrade under the seawall.

The sidewalk east of King Street is in better condition, with less tilting than the remainder of the sidewalk along the Low Battery seawall. The section east of King Street has been reconstructed in the more recent past.

The desired sidewalk design would have a transverse downward slope of approximately ¼ inch per foot of sidewalk width from the seaward edge to the landside edge and have a six inch high granite curb adjacent to the street. In some locations, the existing granite curb and curb inlets are nearly covered with asphalt and need to be raised to their proper full 6-inch exposed height.

Fundamentally, the choices to remedy the tilting sidewalk situation are as follows:

1. Lower the seaward side of the sidewalk as necessary to achieve the desired sidewalk and adjacent six-inch high granite curb design,

2. Raise the landside end of the sidewalk and the adjacent road to achieve the desired sidewalk and adjacent six-inch high granite curb design,

3. Compromise and combine a limited lowering of the seaward side of the sidewalk with a limited raising of the landside end of the sidewalk and adjacent road to achieve the desired sidewalk and adjacent six-inch high granite curb design.

Each approach has its comparative advantages and disadvantages. The first approach of just lowering the seaward end of the sidewalk intuitively seems the least expensive and possibly the least aesthetically pleasing.

With the second approach, it would seem that major reconstruction of the roadway would be a necessity. Additional asphalt overlay with feathering would be necessary on the east bound lane to prevent a greater than 6-inch drop off from the sidewalk level to the street level.

The third approach, a combination of the limited lowering of the seaward end and limited raising of the landside end of the sidewalk and adjacent road, would probably result in the best all around in appearance. The third approach would also result in considerable reconstruction of the adjacent roadway.

Effectively repairing the tilting sidewalks seems to potentially involve considerable interaction with the roadway/drainage/planted boulevard design issues along Murray Boulevard from Tradd Street to King Street.

The tilting of the sidewalk is primarily a long term soil settlement issue. Immediately adjacent to the

occasional storm drain outlets through the seawall, the tilting of the sidewalk is apparently further magnified by localized soil subgrade erosion. Regardless of the final approach taken to rectify the sidewalk/curb/roadway/storm drainage/planted boulevard/settlement design challenges, the tilting of the sidewalk is not a direct seawall issue.

Nevertheless, the estimated costs to remove the existing sidewalks, to raise the existing granite curb, and to build new sidewalks along the seawall from Tradd Street to King Street will be included herein for future budgetary use when considering it as part of a more encompassing Murray Boulevard improvement project.

POTENTIAL LIABILITIES

The Low Battery seawall appears to be well along in its service life span. It is not possible, however, to realistically predict the number of years remaining in its service life span with any degree of certainty. There are just too many unquantifiable variables to account for.

The Low Battery seawall is approximately 0.9 miles long and is a highly redundant structure in the longitudinal direction. Timber batter piles repeat every three feet on center and pairs of timber vertical piles repeat every six feet on center for the entire length of the seawall. By the nature of this redundant design, isolated localities of deteriorated foundation support would not necessarily be a critical condition. The concrete structure of the seawall possesses considerable innate ability to bridge over localized foundation defects.

At present, there are no outwardly noticeable indications of existing or impending major foundation distress. The topographic survey and simple visual sighting along the seaward edge reveal a consistent and level structure. There are no ‘sags” seen when sighting along the seawall. However, the subterranean timber foundation support system fundamentally lacks maintainability and it is a significant portion of the seawall structure.

Barring major storm damage, the probable end of the seawall’s service life will be the result of extensive localized failure of the timber foundation support system. The concrete portion of the seawall eventually would not be able to bridge ever increasing extents of localized foundation failure. Sags, dips, and possible tilting would inevitably result along the length of the seawall. Cracks in the seaward face of the concrete seawall would tend to progress more diagonally upward, rather than horizontally as at present.

The seawall is situated along the northern bank of the Ashley River. The eastern portion is relatively more exposed to severe weather then the western portion. Waves have overtopped the relatively low seawall during major hurricanes. The energy levels of hurricane force wind driven waves are very substantial even in the Ashley River. When directed from a southerly direction, wave action impacts directly upon the Low Battery seawall.

The seawall is immediately adjacent to Murray Boulevard with an extensive neighborhood of homes beyond. Its length and location make the Low Battery seawall a very significant structure for protecting the peninsula.

NECESSARY MAINTENANCE

Currently, the Low Battery seawall is in need of significant amounts of reactive preventive maintenance and lesser amounts of corrective maintenance to extend the service life span to its full potential.

The concern is that if this necessary maintenance is deferred indefinitely, then the seawall's service life span would end prematurely. At that time, either the option for significant amounts of corrective maintenance or the option for the replacement alternative would necessarily have to be considered. For either of those options, the scope of work and the resultant construction costs would dwarf the present maintenance costs.

The opportunity to achieve positive benefits from extensive reactive preventive maintenance presently exists. Nevertheless, timely action is necessary.

To achieve the full potential service life span for the seawall, the timber foundation needs to achieve its full potential service life span. The protection of the structural integrity of the seaward row of timber support piles, both vertical and batter, from the marine boring organisms is of particular importance.

The quality of protection originally provided to the top portions of these seaward piles by the concrete mass of the seawall has deteriorated along much of the length of the seawall. The wide, approximately 95-foot long, horizontal cracks located approximately six feet below the coping are indicative of such ongoing deterioration. The existing soft and weakened concrete located at and below these cracks needs to be removed and replaced with new and stronger concrete.

CONSIDERATIONS OF METHODS

Considerations of some of the methods for the actual construction are an important concern.

Two major areas of consideration are as follows:

1. The integration of construction methods with the tidal states,
2. Construction methods for placing concrete for the repairs.

Because the integration of construction methods with the tidal states is of fundamental importance, two options were explored.

One option would be to install a temporary cofferdam system in discreet intervals. It is envisioned that approximately 200 feet along the length of the seawall would be encompassed by a temporary cofferdam system so that necessary maintenance repairs could be performed without regard to the state of the tides. Upon maintenance completion to that length of seawall, the 200 feet of cofferdam would be removed and reinstalled along the next 200 feet of seawall, and so on. To cover the approximately 0.9 miles of the Low Battery seawall, this process would have to be performed approximately 24 times and would be very expensive.

Another major concern with the cofferdam concept is the ability to install the steel sheet piles into the considerable amounts of stone rip rap present along the base of the seawall. The 200 foot length of the cofferdam system would have to be installed seaward of the toe of the riprap. Installing the closure sections perpendicular to the seawall would be a challenge. It would be necessary to excavate and remove a band of riprap to create a clear path for installing the sheet piles. Driving the sheet pile immediately adjacent to the seawall would not be possible. Some form of a sandbag dam would have to be put in place close to the seawall in order to seal the connection. Continually running sump pumps would seem to be a certainty. Potential claims for additional monies due to the contractor because of the underground obstructions may result in additional costs to the City.

Installing the cofferdam system would be expensive.

The other construction option would be to forgo the cofferdam system and work only during the period around times of low tide. Because considerable noise would emanate from the pneumatic chisels while removing the deteriorated concrete, working around the nightly low tide for months on end would seem to be an unacceptable option. As a result, the contractor would only be able to work at most four hours a day.

Two knowledgeable local contractors were contacted for their insight on these options. They both felt that because the cofferdam system would be more expensive in the long run, the preferred alternative would be working around the tidal states.

A second area of consideration involves the construction method for placing concrete into the zones to be repaired. Potential concrete placement methods include either pneumatically applying concrete "shotcrete" into the excavated voids or pumping concrete into the excavated voids behind forms placed against the seaward face of the seawall. Each placement method would have its comparative advantages and disadvantages.

When repairing concrete, the properties of the repair material would have to be compatible with the existing concrete. Physical compatibility is necessary for a good bond to the concrete substrate; even the level of chloride within the existing concrete may have to be considered. The repair material must adhere to the existing concrete substrate. The color and texture of the repair concrete should match the existing concrete as much as possible.

PROPOSED MAINTENANCE CONCEPT

For the Low Battery seawall, the proposed concept envisioned would utilize the fundamental approach to maintenance. A sketch of the proposed maintenance concept is included in Appendix P. The sequence for the proposed concept envisioned would be as follows:

1. Obtain necessary permits from Federal and State Regulatory Agencies,
2. Work around the tidal states,
3. Temporarily remove stone rip rap located immediately adjacent to the seawall and above the top of the protective concrete veils,
4. Pressure wash the seaward face of the seawall,
5. Remove the zones of soft and weakened concrete along the seaward face of the seawall,
6. Drill the top edge of the concrete veil sections as necessary to reinstall and epoxy grout new steel reinforcing dowels and to increase the amounts of protective concrete cover on the seaward side of the new dowels,
7. Either sand blast or water blast the new interface surface of the existing concrete substrate to achieve satisfactory bond preparation,
8. Install the necessary quantities of repair concrete compatible with the existing concrete,
9. Reinstall the displaced stone rip rap and add supplemental rip rap as necessary,
10. Remove and replace the deteriorated sections of concrete coping along the top of the seawall,
11. Perform other necessary minor concrete repairs to the seaward face of the seawall,
12. Remove and replace the sidewalks and raise the granite curbs extending along the Low Battery seawall from the foot of King Street to Tradd Street as part of a more encompassing Murray Boulevard improvement project.

It is further recommended that a topographic survey be conducted approximately every five years along the top portion of the seawall and at the same stations included in the survey in Appendix J for monitoring purposes.

CONCEPTUAL ESTIMATE OF COSTS

The conceptual estimate of total engineering design and construction costs is $ 5,500,000. Additional details of the estimated costs are included in Appendix Q.

The Stone Masonry Portion of the High Battery Seawall

EXISTING CONDITION

The general condition assessment phase revealed that the seaward seawall (including the stones and

the mortar joints between adjacent stones) and the landside seawall are considered to be in fair overall condition.

On top of the seawall, the flagstone walkway is considered to be in poor overall condition. The flagstone walkway includes the integration of the flagstones, mortar joints between individual stones, the supporting underlying fill, and the bearing of the ends of the flagstones on the seaward seawall and the landside seawall.

It was previously stated in this report that some flagstone slabs are smooth and provide a good walking surface while other flagstones exhibit significant amounts of surface scaling. Some sections of flagstones have previously cracked and were repaired by filling the cracks with mortar. Much of the mortar originally placed between adjacent sections of flagstones no longer exists. In some locations, the variations in elevations between adjacent flagstone sections approached ½ inch.

It was also previously stated in this report that along the seaward face of the seaward seawall, with the exception of the top one or two courses, most of the mortar that was once present in the face of the joints has disintegrated. Regardless, these stones themselves show few signs of deterioration or dislocation.

From the historic research phase, it was learned that the 1893 repairs included an interior concrete wall cast against the landside of the seaward seawall. Consequently, the interior concrete wall was bonded to the landside face of the seaward seawall, which added significant strength to the overall seawall system. Currently, this interior concrete wall prevents the earth fill placed between the seaward seawall and the landside seawall from seeping out through the voided mortar joints.

With no visible indications of structural distress, the seaward seawall system appears to be functioning satisfactorily despite the deteriorated mortar joints.

Previous sections of this report document the observed details of the seawall's physical condition.

POTENTIAL LIABILITIES

As previously stated in this report, the flagstone walkway sometimes provides a "wobbly" sensation, particularly if one's weighted foot was placed near the edge of the flagstone. It is believed that the flagstones were to be partially supported by underlying fill between the seaward seawall and the landside seawall. This fill has subsided over the intervening years and the loss of partial support has contributed to the "wobbly" effect. At present, the flagstones span between the seaward seawall and the landside seawall without the desired intermediary support by the earth fill. The physical conditions of the undersides of the flagstones are unknown. Any attempt to conscientiously monitor the ongoing conditions of the underside of the flagstones would be a major undertaking. It is possible that under certain conditions of heavy loadings, a stone could become overstressed and give way. A sudden failure by an overstressed flagstone could result in physical injury to pedestrians.

By their very nature, the basic materials used in the construction of the flagstone walkway resulted in an imperfect finished walkway. Variations in the top surface of the flagstones are natural and expected. Although reasonable material imperfections add to the historic character of the walkway, deteriorated support and joint conditions for the stones reduce the safety and comfort of the walkway.

The walkway portion of the seawall is traveled by pedestrians of all ages. In some places the walkway presents a very uneven walking surface. Pedestrians, inattentive to the uneven walking surface, could trip and potentially suffer physical injury. Increasing pedestrian safety and comfort would be of great benefit.

Along the seaward face of the seaward seawall, any remaining mortar within the joints between the stones continues to be at risk of erosion due to ongoing tidal and wave action. If the erosion of the remaining mortar continues unchecked indefinitely, then an undesirable localized dropping of stones or a localized downward tilting of the seaward face of the stones could eventually result.

NECESSARY MAINTENANCE

The stone masonry portion of the High Battery seawall warrants special care. This historic seawall structure commemorates Charleston's history to residents and visitors alike.

It seems incongruous that the walkway portion of such a prestigious structure should continue to present a "wobbly" feeling to pedestrians, even occasionally. The walkway portion of the seawall structure and the seaward face of the seaward seawall are in considerable need of reactive preventive maintenance and prescribed preventive maintenance.

The proposed preventive maintenance will not substantially increase the seawall's ability to successfully withstand major hurricanes. Rather, the proposed preventative maintenance is focused on increasing pedestrian safety and comfort as well as addressing concerns regarding the ongoing erosion of the mortar joints along the seaward face of the seawall.

Pointing the seaward face of the stone masonry would entail new mortar being inserted into the voided mortar joints along the seaward face. Mortar in masonry construction is a plastic mixture of materials used to bind masonry units into a structural mass. Mortar serves the following purposes:

1. The mortar serves as a bedding material for the masonry units,
2. The mortar allows the units to be leveled and properly placed,
3. The mortar bonds the units together,
4. The mortar provides compressive strength,
5. The mortar provides shear strength, particularly parallel to the wall,
6. The mortar allows some movement between units,

7. The mortar adds aesthetic value.

Realistically, pointing the stone masonry would serve little value as far as increasing the overall strength of the seawall. Nevertheless, the additional mortar from the pointing process would tend to act as a "plug" and help to protect the existing interior mortar from further erosion due to ongoing tidal and wave action. The additional mortar would help prevent the stones from becoming potentially dislocated.

PROPOSED MAINTENANCE CONCEPT

The sequence for the proposed concept envisioned for maintenance would be as follows:

1. Obtain necessary permits from Federal and State Regulatory Agencies,
2. Temporarily remove groupings of adjacent flagstones,
3. Place geotextile fabric on top of existing earth fill between the seawalls,
4. Add necessary additional fill between the seawalls to provide additional support to the underside of the flagstones forming the walkway,
5. Prepare the bearing/bedding surface for the individual flagstones where they are supported by the seaward seawall and landside seawall,
6. Reinstall the flagstones,
7. Apply mortar to the joints between edges of adjacent stones as necessary to provide transition and limit possible erosion of underlying earth fill,
8. Apply mortar to the joints between the edges of the flagstones stones and the seaward and landside seawall as necessary to provide transition and limit possible erosion of underlying earth fill,
9. Work around the tidal state and point the mortar joints of the seaward face.

CONCEPTUAL ESTIMATE OF COSTS

The conceptual estimate of total engineering design and construction costs is $ 800,000. Additional details of the estimated costs are included in Appendix Q.

The Marina Seawall

EXISTING CONDITIONS

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the seawall is in poor condition. Previous sections of this report document the observed details of its physical condition.

At the investigated station, the portion of the tabby concrete seawall is in a very severely deteriorated condition. The top portions of timber piles and other horizontal timber members that were originally internal to the tabby concrete have been significantly reduced in cross section and/or completely eliminated by marine boring organisms.

The seawall's ability to resist overturning earth pressures and lateral load due to high wind and wave action is seriously compromised. Although the seawall remains in place, it appears to be well past its effective service life span and in some places it appears to be approaching the end of its physical life span.

POTENTIAL LIABILITIES

The Marina seawall is in a relatively protected location when compared to the more exposed locations of the High Battery seawall and the Low Battery seawall. The tabby concrete marina seawall receives shelter from the adjacent marina land located to its west, from the north along Lockwood Drive, and from the neighboring more modern marina structure located to the south. The seawall receives more limited sheltering from the east and the south by an expansive mud bank exposed around the time of low tide.

When propagated from the eastern direction, wave action impacts directly on the eastern face of the Marina seawall. Nevertheless, the Coast Guard pier, located approximately 1/2 mile to the east, acts as a wave barrier to hinder large swells moving up from the mouth of the Ashley River.

In its present deteriorated condition, the tabby concrete seawall provides very limited protection to the adjacent parking lot and a neighboring three story building. Seawater has considerable free access through this deteriorated structure. The low seawall is recurrently overtopped by seawater at times of high tides.

From the historic research phase, it was learned that it was likely constructed sometime during the early to mid 19th century. The tabby concrete seawall is a remarkable historic civil engineering structure.

It served its earlier years as a wharf for a rice mill business nearby.

It seems appropriate that the preservative approach be used for the tabby concrete marina seawall.

CONSIDERATIONS

Replacing the deteriorated horizontal timber reinforcement and extending the top portion of the vertical piles does not seem practical. The seawall would have to be essentially destroyed in order to

replace this internal timber structure. Without the internal timber structure to again resist lateral loads and overturning moments due to earth pressures and hurricane wind driven waves, the tabby concrete structure can at best function as a preserved veneer.

It is envisioned that a secondary seawall be constructed on the landside of the existing seawall. It is also envisioned that the holes, cracks, and crevices in the existing deteriorated tabby concrete seawall be repaired with the historic formulation for tabby concrete. No repair would be made to the deteriorated timber structure internal to the tabby concrete seawall. Nevertheless, the exterior face of the seawall exposed to view would be consistent with its original historic character.

Increasing the loads substantially on the subterranean timber foundation mat of the existing seawall should be avoided. A substantial load increase could potentially destabilize the foundation mat.

With the poor soil conditions at the site, it would seem certain that the new seawall would be constructed with a pile supported foundation. Batter piles are typically utilized in the design of seawalls to resist the horizontal earth pressures from the landside and wind and wave pressures from the seaward side. With the close proximity of the nearly adjacent three story office building, driving batter piles would seem impractical. Without the necessary horizontal clearances, the pile driving rig would not be able to tilt its leads sufficiently to drive the batter piles.

Drilled shafts would seem to be the best remaining choice for the foundation support. Drilled shafts have larger diameters than the more conventional types of piles. The larger diameter of the drilled shafts could effectively provide the necessary horizontal restraint to the seawall that would have been provided by the batter piles. The larger diameter drilled shafts would act as a vertical cantilever beam to resist the horizontal earth pressures from the landside and the wind and wave forces from the seaward side as well as a conventional pile to resist the vertical loads from the new concrete seawall. The drilling operation used instead of a pile driving operation would help to reduce the nearby vibration levels.

The proposed new landside seawall would have a foundation supported on drilled shafts. The existing tabby concrete seawall would continue to be supported on a timber raft. With the differing types of foundations, it would be best if a discreet separation existed between the existing and the new seawalls to limit potential differential settlement issues.

PROPOSED REPLACEMENT CONCEPT

A sketch of the proposed concept is included in Appendix P.

The sequence for the proposed replacement concept envisioned would be as follows:

1. Obtain necessary permits from Federal and State Regulatory Agencies,
2. Remove the existing asphalt parking surface immediately adjacent to the landside of the

existing seawall,

3. Carefully excavate down on the landside of the existing seawall to remove any existing large obstructing debris in the earth fill,
4. Power wash, water blast and/or sand blast the seaward and the landside faces of the existing seawall to prepare it for the necessary tabby concrete repairs,
5. Work around the tidal state and repair the seaward and landside faces of the existing tabby concrete seawall,
6. Drill the shafts, set in place the steel reinforcing cages, and place the concrete into the shafts,
7. Remove from the site the earth excavated by the drilling operation,
8. Place the steel reinforcing bars and pour the concrete for the pile-cap portion of the new landside seawall,
9. Place the steel reinforcing bars and pour the concrete for the vertical wall portion of the new landside seawall,
10. Remove the temporary forms between the new and existing seawalls and fill the narrow void between with crushed stone fill to maintain a bond breaker,
11. Replace the earth fill and restore the asphalt parking lot.

CONCEPTUAL ESTIMATE OF COSTS

The conceptual estimate of total engineering design and construction costs is $ 1,200,000. Additional details of the estimated costs are included in Appendix Q.

PHASE IV GENERAL CONCLUSIONS

The recommended sequence of priority among the historic seawalls for maintenance is as follows:

1. The Concrete Extension of the High Battery Seawall,
2. The Low Battery Seawall,
3. The Stone Masonry Portion of the High Battery Seawall,
4. The Marina Seawall.

The Concrete Extension of the High Battery Seawall

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the seaward seawall and the concrete walkway are in poor condition. The seaward seawall portion and the concrete walkway portion appear to be at or near the end of their respective service lives. In its present deteriorated condition, it seems doubtful that the concrete extension of the High Battery seawall could successfully withstand the direct onslaught of a major hurricane without substantial damage. If this seawall should be significantly breached during a major hurricane, hurricane driven waves could propel flood waters well into the southeastern portion

of the peninsula.

In consideration of the extent, nature, and location of the deterioration it appears that the replacement alternative would be both the more economic and practical choice.

The conceptual estimate of total engineering design and construction costs is $1,800,000.

The Low Battery Seawall

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the exposed portions of the seaward face of the concrete seawall range from poor condition to fair condition. The tilting of the sidewalk is primarily a long term soil settlement issue, not a direct seawall issue. The Low Battery seawall appears to be well along in its service life span. It is not possible, however, to realistically predict the number of years remaining in its service life span with any degree of certainty. The seawall is immediately adjacent to Murray Boulevard with an extensive neighborhood of homes beyond. Its length and location make the Low Battery seawall a very significant structure for protecting the peninsula. Currently, the Low Battery seawall is in need of significant amounts of reactive preventive maintenance and lesser amounts of corrective maintenance to extend the service life span to its full potential.

The conceptual estimate of total engineering design and construction costs is $5,500,000.

The Stone Masonry Portion of the High Battery Seawall

On top of the seawall, the flagstone walkway is considered to be overall in poor condition. The flagstone walkway includes the integration of the flagstones, mortar joints between individual stones, the supporting underlying fill, and the bearing of the ends of the flagstones on the seaward seawall and the landside seawall. The deteriorated support and joint conditions for the stones reduce the safety and comfort of the walkway. The stone masonry portion of the High Battery seawall warrants special care. The historic seawall structure commemorates Charleston's history to residents and visitors alike. The walkway portion of the seawall structure and the seaward face of the seaward seawall are in considerable need of reactive preventive maintenance and prescribed preventive maintenance. The proposed preventive maintenance will not substantially increase the seawall's ability to successfully withstand major hurricanes. Rather, the proposed preventive maintenance is focused on increasing pedestrian safety and comfort as well as addressing concerns regarding the ongoing erosion of the mortar joints along the seaward face of the seawall.

The conceptual estimate of total engineering design and construction costs is $ 800,000.

The Marina Seawall

The general condition assessment phase revealed and the subsequent detailed investigation phase further substantiated that the seawall is in poor condition. The seawall's ability to resist overturning earth pressures and lateral loads due to high wind and wave action is seriously compromised. The seawall appears to be well past its effective service life span and in some places it appears to be approaching the end of its physical life span. The Marina seawall is in a relatively protected location when compared to the more exposed locations of the High Battery seawall and the Low Battery seawall. However, in its present deteriorated condition, the tabby seawall provides very limited protection to the adjacent parking lot and a neighboring three story building. To alter this, it is envisioned that a secondary seawall would be constructed on the landside of the existing seawall. It is also envisioned that the holes, cracks, and crevices in the existing deteriorated tabby concrete seawall be repaired with the historic formulation for tabby concrete. No repair would be made to the deteriorated timber structure integral to the tabby concrete seawall. Nevertheless, the exterior face of the seawall exposed to view would be consistent with its original historic character.

The conceptual estimate of total engineering design and construction costs is $ 1,200,000.

Combined Conceptual Estimate

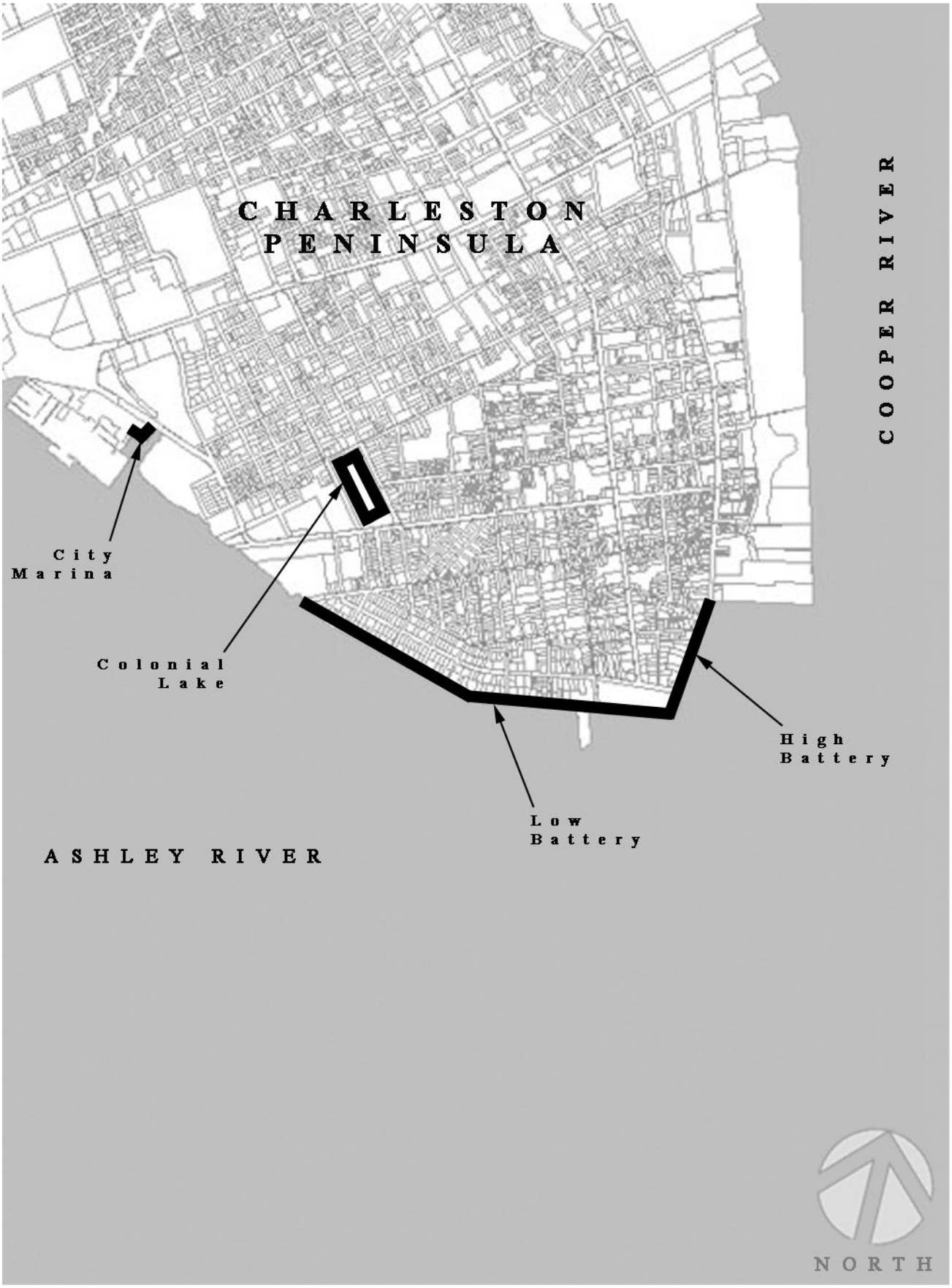
For the concrete extension of the High Battery seawall, the Low Battery seawall, the stone masonry portion of the High Battery seawall, and the Marina seawall the combined conceptual estimate of total engineering design and construction costs is $ 9,300,000.

Written by:

John C. Sheridan IV, P.E.

Reviewed by:

Craig M. Bennett Jr., P.E.



Location Map