II. Seawall Assessment

Katherine Andruchuk, Seth Lizotte

Seawall Assessment Contents

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Introduction

The Wood Island Life Saving Station is protected by two concrete seawalls located on the north and south faces of the building. Over the past century the seawalls have been damaged repeatedly by tidal surge and wave action. The damage done to the seawalls has made it less of a protective barrier for the building and more of a potential hazard for those who visit the island. If the seawalls continue to be left alone, it's only a matter of time until full deterioration occurs. The south face seawall has previously been repaired by Shotcrete Systems International, Inc. as a temporary fix but continues to deteriorate. This report is meant to outline some of the feasible options to remediate these problems.

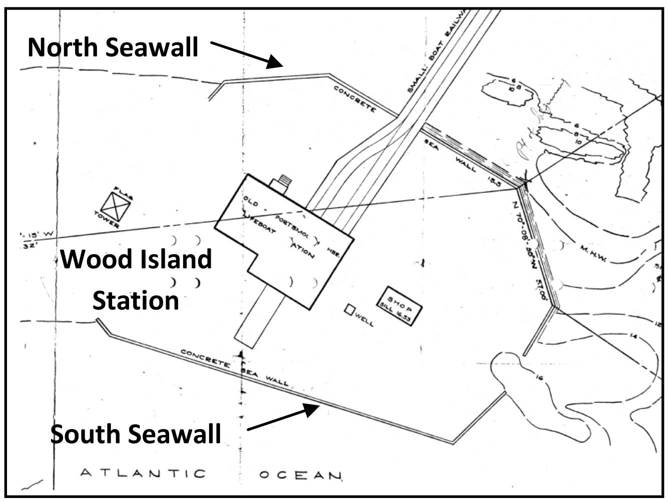

Figure 18 Seawall (Taken from a topographic map dated 1955)

The options described include: leaving the seawall in its current state and letting nature take control, removing the wall completely, or capping over the current wall. Other options include demolishing the wall and using it as backfill for a brand new cast in place seawall or as backfill for a precast wall. Some of the issues faced with these designs include getting the materials required for these options to the island. Another issue takes into account where to put the materials once removed.

Conditions Assessment

A site visit was performed on March 5, 2009 to determine the extent of damages to the existing seawall structures on Wood Island. During this visit, Duncan Mellor, P.E. helped in our observations and field tests performed on the existing structure. The following is a conditions assessment for both the north and south seawalls.

South Seawall

Undercutting

One of the initial observations made of the south seawall was the signs of undercutting by wave action.

Figure 19: Undercutting of South Seawall

Figure 20: Closer Image of Undercutting of South Seawall

Undercutting has occurred on the structure since no foundation was originally constructed for the seawall. As seen from visual inspection, it was rather cast in place on the island and anchored into the bedrock. As weathering occurs over time, waves come in contact with the bottom of the seawall and wear away at the base. Weathered segments of the wall are then carried away, undercutting the seawall as a whole and thus weakening the overall stability of the structure.

Shotcrete

By visual inspection, the entire two hundred and fifteen feet of wall have been previously covered with a shotcreted face. This cap consisted of sparse fiber reinforcement in the paste, a 1/8” mesh reinforcement cage which overtopped the existing wall, and ranged in thickness from less than half an inch in some areas to as much as 2 inches in others. From observation, it seems that the previous contractor filled holes present on the surface of the existing structure with gravel found on the island prior to capping, to fill in voids, see figure below.

The chain and hammer test was used on the cap to determine the bonding of the shotcrete cap with the original wall. The chain test is where a large chain is dragged over the horizontal surfaces to detect a change in pitch where voids would be present under the surface. From the chain test it was found that the top horizontal face of the wall was not bonded for the entire length of the wall. The hammer test is done on vertical faces of the wall, similar to the chain test. It can determine voids present under the surface through changes in pitch as one bangs on the outside of the wall. The area of concern for bonding was found to be between the two mid-level weepholes on the wall extending to 10 feet on either side of them. In this area, a hollow sound was heard which suggests the cap was not bonded.

Figure 21: Damaged Top Section of South Seawall

Figure 22: Un-bonded Shotcrete Cap of South Seawall with Gravel Fill

Figure 23: Shotcrete Cap Not Bonded along face near Weepholes

Weathering

To determine the internal state of the existing south seawall, a chisel and hammer were used to expose concrete further into the structure. Since the most noticeable signs of weathering had occurred surrounding the weepholes, it was determined this was the best location to see the full extent of the damages. Using the chisel and hammer, a hole was made into the face of the wall with minimal effort. During chiseling, visual signs were seen of the degradation of the paste which bonds to the aggregates and provides strength. The paste crumbled into a sandy mixture and thus provided no strength to the wall; see the figures below. To examine various portions of the south wall, other holes were made which revealed the same results. It seems that not only the exterior of the wall was susceptible to weathering and conditions, but as freeze thaw and Alkali-Silica Reactions occurred, weathering moved internally and has severely weakened the structure.

Figure 24: Chiseling into Original Wall near Weephole Weathering

Figure 25: Weathering of Paste as seen through Chiseling

Drainage

Drainage is essential to release hydrostatic pressures behind the wall as waves and rainfall stagnate behind the structure. To provide drainage, seawalls have weepholes located along the structure to allow dissipation of stagnant water. Weepholes can be anything from designed cracks along the wall which allow water to flow through them, to piping which penetrates the width of the wall. When observing the south seawall there was only four weepholes along the entire length. Two were located about 3 feet from the toe of the structure and two were located at the toe of the structure. Due to the inadequacy of the drainage, it conceivably led to the erosion of the backfill as well as further freeze thaw cycling as retained water would be in constant contact with the structure.

Under Duncan Mellor's guidance from experience on previous projects, it is recommended that weepholes be present in 8 foot intervals horizontally and with 3 foot vertical spacing along the entire length of the wall.

Figure 26: Weephole located halfway up on South Wall

Figure 27: Weephole at toe of South Wall

Backfill

Backfill is added behind seawalls to allow for structure support against wave action as well as providing adequate drainage of waters. It allows water retained behind the wall to migrate towards areas of drainage. Due to the inadequacy of the drainage on the south wall, it was observed that the backfill had significantly eroded along the entire length. Since the wall was shotcreted well after the initial structure had been constructed, a visible line could be seen where the shotcrete had once come to the interface of the backfill which resided behind the wall. Seen in the figure below is a distinct line which indicates the initial position of the backfill, and as seen, this line is now over a foot above the now residing backfill. This clearly shows that the backfill which is integral to the stability of the structure has and is continuing to erode.

Figure 28: Signs of Backfill Weathering

Tides

From collected tide data it suggested that high tide would not come within 5 feet of the toe of the existing structure. Upon the March 5, 2009 site visit, shortly after a recent snowfall, marks of the high tide could be seen in the melting of the snowfall. Seen in the figures below is that high tide does come into contact with the bottom of the wall and that this would warrant concern of continuing undercutting of the structure and weathering.

Figure 29: High Tide Marks on South Seawall (1)

Figure 30: High Tide Marks on South Seawall (2)

Nautical Maps

The color on a nautical map is a way of highlighting various features. Pale gold is used for land areas, white is used for water areas, pale blue is used for shallower waters, and green is used for areas that are submerged during some tidal stages and not submerged during others (Hoff, 2009). As seen in the figures below, Wood Island is primarily in the green area, having the Life Saving Station within the pale gold area. Having the majority of the island lying in the tidal area, and due to visual signs of the tidal reaches, this shows the clear importance of a seawall in serving as protection for the building during these tidal stages and during large storm events.

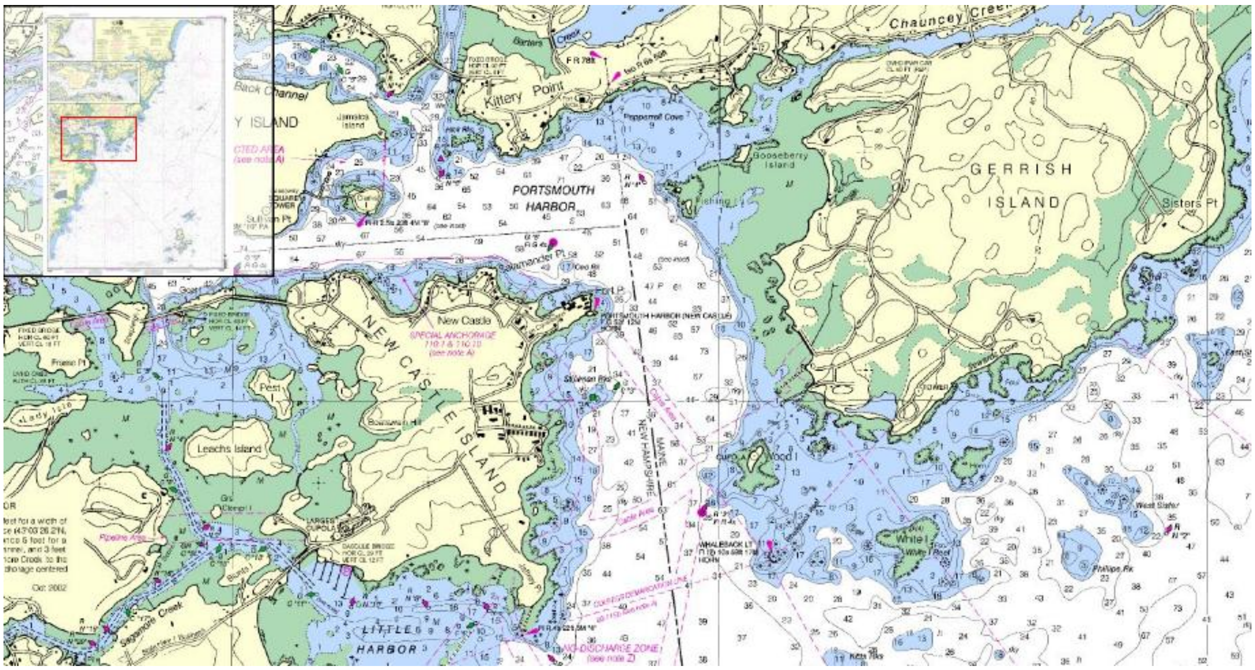

Figure 31: Nautical Map (1) (Administration, 2009)

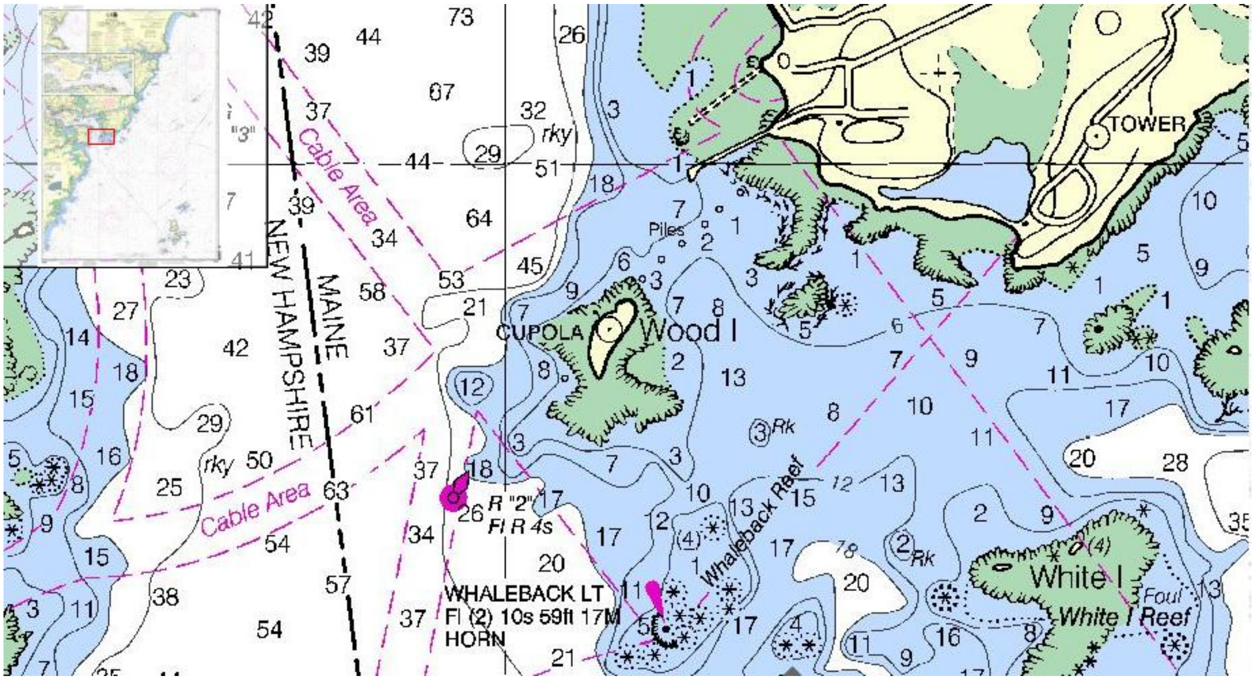

Figure 32: Nautical Map (2) (Administration, 2009)

Sulfate Attack

Seen in the figure below is the leaching of sulfates or salts, which were deposited from the seawater, from the face of the South Seawall. These sulfates may have contributed to the degradation of the paste of the wall, but furthermore have led to severe deterioration of the reinforcing steel mesh of the shotcreted cap. Further consideration must be paid to sulfate attack in the rehabilitation efforts due to reinforcement of a new structure as well as the use of tie anchors which would experience corrosion.

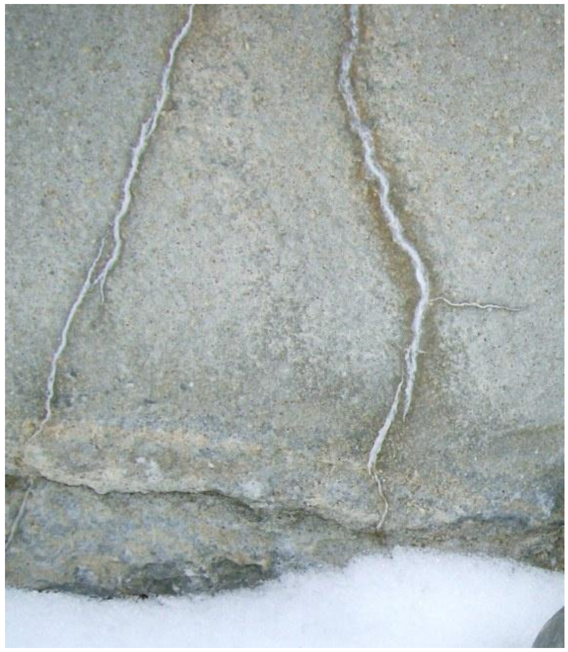

Figure 33: Leaching of Sulfates from South Seawall

North Seawall

Undercutting and Overturning

The north seawall shows significant signs of undercutting due to wave action. Since the seawall was originally placed directly on top of the exposed bedrock, and no footing exists, waves which come into contact with the toe of the seawall slowly erode and wash away the toe of the wall; undercutting the structure. Seen in the figures below are the combinations of undercutting at the toe and freeze thaw deterioration. As undercutting occurs, it poses large risks to the stability of the structure.

Figure 34: Undercutting of North Seawall (1)

Figure 35: Undercutting of North Seawall (2)

The combination of undercutting and other weathering events on the seawall cause a change in the stability of the structure. As the foundation wears away and wave force pounds at the structure, the wall begins to collapse due to its displaced center of gravity and continual battering. Seen in the figures below is a section of the seawall which is experiencing this local instability due to undercutting and wave action. As parallel freeze thaw cracks move across the face of the wall, the cracking allows for internal degradation of the concrete. As seen below, the top portion of the structure is independent of the bottom and has begun to overturn. Since the structure has cracked and acts in independent fashions, it allows for the horizontal and vertical displacement of the structure as it fails and sooner or later will fall over.

Figure 36: Horizontal Displacement and Overturning of North Seawall (1)

Figure 37: Horizontal Displacement and Overturning of North Seawall (2)

Construction Joints

It seems that the north wall was constructed in segments. At locations approximately every 10 ft horizontally it was observed that construction joints lie where one section ended and another began. These construction joints may have served as expansion and contraction joints or rather as paths for drainage, but have since become a localized area for freeze thaw action. As water penetrates the cracks and then freezes, it expands and then puts stresses in between the segments. This pressure slowly forms cracks and as seen in the figure below is the primary region where the most severe weathering has occurred.

Figure 38: Weathering at Locations of Construction Joints along North Seawall

Freeze Thaw Cycling

Freeze thaw action seems to originate at the construction joints. From there, the water works its way into the internal structure of the wall and forms cracks parallel to the surface. These cracks are vividly seen in the figures below and contribute to the overall weakening of the stability of the structure. As the water expands internally, the induced internal stresses break apart the concrete. Freeze thaw is more obviously seen as the figures below show faces of the wall which are coated in ice from the nearby waters.

Figure 39: Freeze Thaw Action on North Seawall

Figure 40: Visible Freeze Thaw on North Seawall

As water penetrates into the wall, its stresses can expand around aggregates and in turn form voids around the aggregates. Since native aggregates on the island were most likely used in the walls construction, they were largely varying in size and posed larger areas for water to encompass. Seen below, the water infiltrated the surface of the wall at one point and went through freeze thaw cycling around the aggregate located in the picture. This in turn leads to un-bonding of the aggregate and a sever decrease in the structures overall strength. The voids present around the aggregates also are typical of Alkali-Silica Reaction present in the structure as internal stresses develop from ASR.

Figure 41: Un-bonding of Aggregate due to Freeze Thaw in North Seawall

Alkali Silica Reaction

Alkali-Silica Reaction (ASR) occurs between the hydroxyl ions in the alkaline cement pore solution present in the paste of the concrete and reactive forms of silica in the aggregate. When this occurs, a gel-like substance is formed from the reacted paste and absorbs water; inducing internal stresses in the concrete (Consultants, 2005-2009). Duncan Mellor explained that the cracking seen from ASR is not parallel in structure similar to that of freeze thaw action, but forms spider web cracks similar to that seen in the figure below taken from the north seawall. These cracks were seen along the surface of the north seawall and gave reason for further testing to occur within the laboratory to confirm that ASR was occurring. Also, the geometry of the cracks which originated at the construction joints, due to freeze thaw, showed signs of ASR in that at the corners they would curve upward. This was noted as a sign of ASR by Duncan Mellor during a site visit.

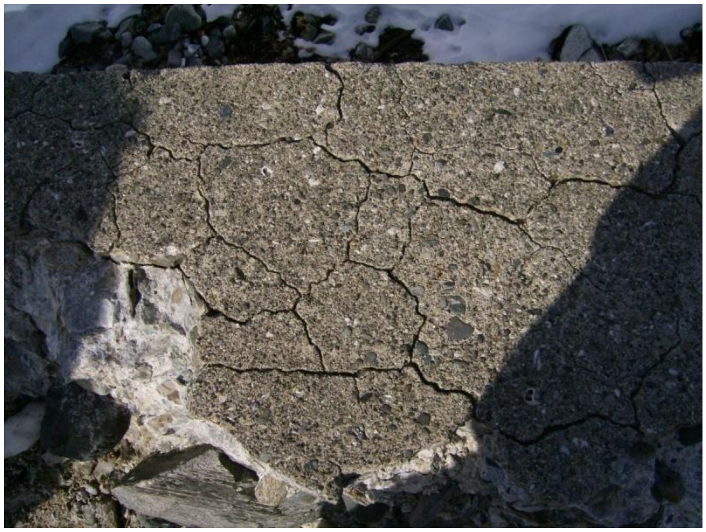

Figure 42: Spider Web Cracking Signs of ASR along North Seawall

Drainage

Drainage paths were seen on the north wall as cast in place holes rather than weepholes as the south side had. The drainage holes were about 3 inch by 3inch and had one located at the bottom and one about halfway up the wall for every segment. As seen in the figure below, these were also sites for freeze thaw action to penetrate and form the typical parallel cracking across the walls face.

Figure 43: Cast in Place Drainage along North Seawall

Backfill

Backfill is used for two purposes with a seawall. It provides stability from overturning when waves come into contact with the structure and also serves as a free draining material so that water pressure does not build up behind the structure. In some areas it was observed that backfill had once been a few feet up behind the existing north wall, but due to weathering, most areas had no backfill present. This leads to further stability issues since there is no longer the mass of the backfill present behind the wall to resist the wave forces.

Tie Anchors

Vertical tie anchors were seen where sections of the wall had collapsed and been washed away. The tie anchors seen, extended about 2 ft vertically from the bedrock and were used to provide stability to the foundation of the original wall. In areas where large portions of the wall were missing, it was seen that tie anchors were spaced about 10 ft horizontally along the length of the wall. Seen in the pictures below are the tie anchors, and serious corrosion has occurred due to the sulfate content of the nearby seawater.

Figure 44: Corrosion of Tie Anchors along North Seawall (1)

Figure 45: Corrosion of Tie Anchors along North Seawall (2)

Analysis Conducted

Laboratory Studies

To determine the extent of weathering and other damages to the existing seawall on Wood Island, various laboratory tests were performed and observations made on a sample taken from the northern seawall which faces Portsmouth Harbor. The following sections include descriptions of the possible issues studied and a narrative of the observations made:

Alkali Silica Reaction

Alkali-Silica Reaction (ASR) is a reaction between the hydroxyl ions in the alkaline cement pore solution present in the paste of the concrete and reactive forms of silica in the aggregate. When this occurs, a gel-like substance is formed which expands in volume by absorbing water present in the concrete creating high expansive stresses usually ranging from 250 to 300 psi. This expansion results in failure, through cracking, in the concrete and in turn structural deterioration of the structure (Consultants, 2005-2009).

One of the more common tests to detect ASR involves the use of uranyl acetate, a radioactive uranium compound. To perform this test, a freshly fractured face must be used to take the uranyl acetate compound, and to do so a hydraulic compression testing machine was used to crush the sample. Once a fractured face was obtained, the sample was taken to the laboratory and uranyl acetate was added as a liquid solution to the fractured face. Excess solution was rinsed off, and then the sample was examined in a dark room with the use of black lights. Under a black light, the gel formed from ASR fluoresces much more brightly than the cement paste due to the greater concentration of alkali and, therefore, the uranyl ion present in the gel.

From observations, ASR was present in the sample, yet to a degree which seemingly was of minimal concern to the integrity of the structure. ASR was also observed in a cut and polished section of the sample and was seen as dark rings which surround some aggregate faces showing the damage due to portland cement expansion.

Figure 46: Hydraulic Crushing Machine

Figure 47: Core Sample after Fracture

Figure 48: Addition of Uranyl Acetate Solution

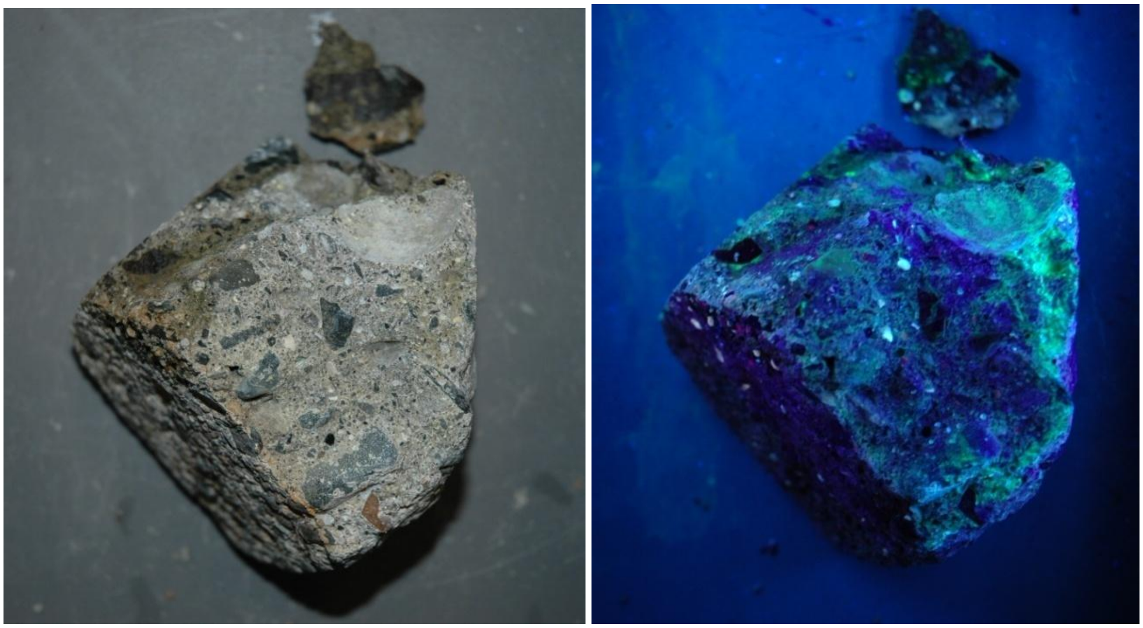

Figure 49: Sample Showing before and after with ASR Fluorescence under Black Lights

Since a sample was used, in considerably small volume compared to the entire structure, it is hard to correlate to the entire structure. More samples would be needed to determine the exact extent of the acceleration of ASR within the entire system, though observations suggest minimal threat.

Cutting and Polishing of Sample

To prepare the sample for other microscopic observations, it had to be cut into various sections using a concrete chop saw. From there, the faces were polished using a concrete polishing turntable. The following figures depict this process:

Figure 50: Concrete Saw

Figure 51: Cut Concrete Sample (1)

Figure 52: Cut Concrete Sample (2)

Figure 53: Polished Sample Concrete Face

Carbonic Acid

Carbonation is a process of weathering which reduces the alkalinity of the concrete it reacts with. During this process, carbon dioxide in the air dissolved in any moisture on or underneath the surface of the concrete forms carbonic acid. The carbonic acid then migrates into the structure of the concrete, forming cracking and reducing its alkalinity, and hence its ability to protect reinforcement from corrosion.

Though reinforcement is not present in the seawall being observed, it is important to note the observations for the possible addition of reinforcing bar in proposed rehabilitation efforts to determine the susceptibility of certain systems under the project conditions. Also noted, carbonation usually strengthens the concrete surfaces, increases wearing resistance, and makes it less permeable.

Evidence of carbonation during observation of the cut and polished sample can be seen with a detectable brownish haze which envelopes the exposed surface of the specimen. Cracking due to carbonation was not observed and it seems that the long term effects would be negligible for rehabilitation. It may actually serve to provide a water barrier if a capping system over the existing structure is chosen.

Freeze Thaw Cycling

Deterioration of concrete due to freeze thaw cycles may occur when the concrete becomes saturated as the pores fill with water. When the water filled pores are then exposed to low temperatures, it then freezes, and if there is no space for expansion, the water causes internal stresses within the concrete. If the stresses cannot be compensated by the concrete structure, the concrete forms cracking to allow for expansion. The cracking during freeze thaw cycling occurs parallel to the external face because as moisture penetrates the face, it does so in layers corresponding to the surface of the face. Each successive freeze thaw cycle buries deeper into the structure of the concrete and forms subsequent larger cracking and deterioration.

Evidence of freeze thaw cycling could be seen when a cut and polished section was examined under the microscope. While observing the sample, large cracks were seen extending across the plane parallel and close to the face of the sample. Though it exhibited signs of cycling, the damages seen within the sample seemed minimal with respect to the overall integrity of the structure. Other typical signs the sample exhibited during a site visit were small chunks which had come off of the structure. This could be probable freeze thaw cycling or even impact loads from waves.

To prevent concrete from freeze thaw damages, concrete is air entrained to allow air voids for moisture expansion. Upon observation, it was hard to detect that the sample was air entrained or if small voids were left due to not being fully compacted when placed. To prevent further freeze thaw damages, air entrainment will be examined for the rehabilitation proposal.

Internal/External Sulfate Attack

Internal/External Sulfate Attack occurs when water containing dissolved sulfate, such as oceanic saltwater, penetrates the concrete. Evidence of external sulfate attack can be seen on a cut and polished section under the microscope at the reaction front. This occurs near the face of the sample where moisture can penetrate. Internal sulfate attack can be seen as saltwater penetrates the pores of the internal concrete structure. Sulfates attack the composition of the paste and results in an overall loss of concrete strength and bond between the cement paste and the aggregate.

Due to the environmental considerations of the seawall, it was important to search for signs of sulfate attack on the seawall. Under observation, no signs of sulfate attack were apparent, though considerations should be made to ensure rehabilitation efforts account for sulfate content in the accompanying waters as far as reinforcement.

Assumptions

The aforementioned laboratory studies do reveal quite a bit about the extent of weathering on the existing structure, though it is impractical to extrapolate the results from a small sample to the whole of the structure. If an alternative is chosen which integrates the existing seawall, it should be known that the studies in this assessment address a small piece of the entire wall and other sections of the wall may exhibit deterioration due to weathering that is of a much larger extent than appeared in the sample.

Solutions Considered

Precast

Precast alternatives were considered due to ease of construction, durability, and aesthetics.

Redi-Rock System

One option considered was Redi-Rock's Big Block ® seawall construction. This was the system recommended by the supplier for our environmental conditions and used blocks measuring 18” high, 46” wide, and 36” deep, and weighing 2,400 lbs each. To prevent hydrostatic pressure build-up and possible freeze thaw damages associated with water retention behind the wall, it was recommended to backfill with 3’-4’ of porous fill (gravel and crushed stone). This system would need to be set upon a 8”-12” thick by 36” deep footing to be poured for a level working area and would need to be tie anchored back into the existing bedrock to prevent a sliding failure. The estimated cost for Big Block ® materials came to be $210,000 which included the delivery and placement. Demolition of the existing seawall, pouring of a new footing for the precast seawall, and backfill material was not considered in this cost estimation.

Tectonics, Inc.

A series of conversations were held with Robert G. Armando, President of Tectonics, Inc. regarding precast alternatives for the seawall rehabilitation efforts. During these conversations Mr. Armando assisted the Wood Island Group in determining relative costs and construction efforts needed to properly address the weathering, deterioration, and location constraints of the existing structure. A survey using Google Earth Pro measured the existing wall at 420 ft total, but it was under his recommendation to address a 600 ft wall. This would more effectively protect the Life Saving Station from all directions from wave forces by encompassing the buildings perimeter. The 600 ft alternative was not considered due to permitting issues which would involve new construction of approximately 180 ft which is beyond the existing 420 ft that exist. Due to other permitting issues surrounding environmental impacts, it was decided that the best process for precast would be to place a new wall in the same location as the existing wall.

Construction Logistics

The ideal installation would have the existing wall grinded down for later use as backfill material which would save on the cost of demolition and removal of that material. In turn a new cast in place slab pinned to bedrock and a precast superstructure bolted to the slab would be used in the location of the existing wall. The precast superstructure can be plant cast in the area and transported to the site using amphibious barges for erection. Once the pre-casting is complete, the demolition of the existing wall and construction of the base slab can proceed together. The precast structure can be erected no sooner than 7 days following the base slab due to concrete curing time. The total estimated construction time, given moderate weather conditions, would take 6 weeks from the time the precast components are cast to the final placement of the seawall.

Once the existing seawall is grinded and placed out of the way for later use as backfill material, the construction of the new footing may proceed. The footing would consist of an 18” thick and 42” deep concrete slab cast in place with the precast wall sitting 8” back from the face of the slab. The footing placement will be superseded by grinding into the bedrock to provide a level surface for the cast in place footing to be anchored into. It is recommended that 4,000 to 5,000 psi concrete be used for all concrete components of the seawall and that waterproofing additives be used to prevent freeze-thaw damages. Concrete placement can be done through the use of amphibious barges and either mobile-mix concrete trucks or redi-mix concrete trucks utilizing a pumping system. Since amphibious barges are needed to be able to access the island, work must be scheduled around changing tides at the island for accessibility constraints. Placing the footing partially into the existing bedrock will effectively minimize the chance for undercutting and help in wave dissipation prior to coming in contact with the seawall face.

The footing will have 1” to 1 ½” tie bolts anchoring into the bedrock every 8 ft and made of high strength coil bolt inserts. Steel plates will tie into buttresses located at the base of the superstructure which will tie into the rebar present in the footing for later post-tensioning. All reinforcement which will be exposed to weathering would be epoxy coated and or covered with a bituminous material to reduce corrosion. Finally the Redi-Rock ® Big Block system would be erected upon the footing and post tensioned for stability, following the desired footing curing time. The desired wall height is 6 ft tall on the south wall and 8 ft on the north wall due elevations.

Figure 54: Redi-Rock ® Big Block System (Redi-Rock Retaining Wall Series, 2009)

With concern to drainage, weepholes will be cast integrally with the footing, every 8 ft horizontally. Vertical construction joints present in the precast block system provide for vertical drainage mode pathways. To direct water towards the weepholes present in the footing, perforated pipe will lie behind the wall and direct water toward the weephole locations. The perforated pipe will be covered with filter fabric to prevent clogging and then backfilled with the grinded existing seawall to act as a porous free draining media. With the entire proposed precast system in place, it is guaranteed a 50 year design life.

Cost

The conceptual costs associated with the precast alternative are representative of area suppliers and potential subcontractors as used by Tectonics, Inc. The cost is also a function of the weather and tidal surges in the area since work performed is dependent on tidal cycles when using the amphibious barges. The precast system is approximately $1500 per linear foot to cast in place the footing with tie anchors and bedrock grinding, grind the existing seawall for backfill material, and place the new precast seawall. This amounts to approximately 1/3 of the cost for the precast elements and 2/3 the total cost for demolition, an anchored base slab with drainage system, tie backs, and backfill.

The cost to construct a precast seawall in place of the existing 420 ft length of wall would be about $650,000, and to construct the entire 600 ft recommended would be $900,000. A 15% contingency to cover the possibility of extraordinary weather events and the possibility of storm damage during construction is recommended, bringing the 420 ft recommendation to $748,000.

Capping System

The option to place a reinforced cap surrounding the existing wall was researched and found to be ineffective due to the state of deterioration of the existing seawall. The option looked into was to chip away the outer deteriorated concrete and place a reinforced cage of stirrups which would be drilled and placed integrally with the existing wall then pour a concrete cap around this system. The option to spray a hydrophobic foam surrounding the face of the existing wall was also looked into which would prevent further weathering of the existing wall and allow the new cap to act independently from the existing seawall. Although this would help stabilize the existing wall, it was deemed ineffective to serve as a long term solution and thus was abandoned. Below are the recommended design components for a capping system:

Mix Design for Capping Solution

Determined from the American Concrete Institute (ACI) Code, it was found that if minimal steel was used for the reinforcing cage that a minimum of 1 ½" cover be provided for the reinforcing bars when exposed to weather (ACI 7.7.1) (Institute, 2008). This would give a minimum of 3" of concrete to be placed surrounding the existing structure to allow for proper protection for the reinforcing steel. Through the recommendation of Dr. Gress and from guidelines present in the Portland Cement Association Code, the concrete placed was recommended to be greater or equal to 4000 psi concrete with a water-cement ratio less than or equal to 0.4 . Due to sulfates present in the ocean, high quality Type II cement must be used. Air entraining admixtures must be added to have the air content be greater or equal to 6% (Steven H. Kosmatka, 1994).

Reinforcing Steel for Capping Solution

The reinforced cage which would surround the existing seawall would be composed of stirrups tied together by longitudinal reinforcing bars. For the stirrup and tie hooks it was recommended by the ACI Code to have them be embedded into the existing seawall at least 5 bar diameters. Since no. 5 bars were to be used, this would be to have them embedded 3-1/8" into the existing wall (ACI 7.1.3) (Institute, 2008). The maximum spacing of the stirrups in the horizontal direction would be determined from the actual volume of new concrete cast in place. The minimum ratio of horizontal reinforcement area to gross new concrete area must be 0.0020 for reinforcing bars 5 or smaller with a yield stress not less than 60,000psi (ACI 14.3.3). The vertical spacing of reinforcing bars to tie these stirrups together is determined from the volume of new concrete cast also. The minimum ratio of vertical reinforcement area to the gross new concrete area must be 0.0012 for reinforcing bars 5 or smaller with a yield stress not less than 60,000psi (ACI 14.3.2). Both the vertical and horizontal spacing is limited, however, to be a maximum of 18" (ACI 14.3.5).

The aforementioned criteria were guidelines considered for the capping system, and since it was found to not be feasible, further detail was not pursued.

Demolish Wall and New Wall Cast in Place

Tectonics, Inc.

A series of conversations were held with Robert G. Armando, President of Tectonics, Inc. regarding the logistics of a cast in place seawall. During these conversations, it was recommended to go with a battered wall to resist the wave forces associated with the area, since through his experience straight walls with tie backs were found to be more expensive and less effective. The recommended dimensions were to have a 36” base width tapering to a 12” top width with an overall height of 72”.

Construction Logistics

The foundation would remain the same as aforementioned in the precast alternative, with the exception of the addition of weepholes spaced at 8 ft on center located 48” from the base of the seawall and weepholes located at 8 ft on center at 12” from the base. The weepholes proposed would not be formed but rather composed of PVC piping at least 1-1/2” in diameter. Due to the size of the wall, the base of the formwork would need to be heavily reinforced to prevent blowout. The formwork would allow for construction joints located approximately every 10- 15 ft and would be flexible joints with a water seal to protect against freeze thaw damage. It was recommended to use C or U shaped stirrup ties for the reinforcing cage and have the bar size be a minimum of a No. 5 bar. All reinforcing bars used would need to be epoxy coated to prevent corrosion from the sulfates present in the sea water. Since the structure would not be post tensioned as with the precast alternative, it was recommended to have the tie anchors present in the footing extend at least 18” from the footing slab to allow for appropriate stability. The tie anchors would be 1-1/2” diameter and would be grout anchored into the bedrock. The concrete used would need to be a 4-5 ksi mix with a low water cement ratio less than or equal to 0.4. This would be made of Type II cement to prevent sulfate interaction with the reinforcing bars and would have an additive for waterproofing. The cast in place wall, once formwork is removed, would later be backfilled with the grinded existing wall to provide for a free draining material to prevent hydrostatic pressure. The design life for the structure would double that of the precast structure due to corrosion issues with the post-tensioning steel used and would be nearly 100 years.

Cost

The conceptual cost associated with the cast in place alternative is representative of area suppliers and potential subcontractors as used by Tectonics, Inc. The cost is also a function of the weather and tidal surges in the area since work performed is dependent on tidal cycles when using the amphibious barges. The cast in place wall is approximately $1000 per cubic yard poured. This cost includes all transportation and a placement cost associated with the wall itself, and does not include the footing. Included in this cost are the formwork, labor, reinforcing bar, concrete, barges, and other associated cost with the placement of the concrete. The total volume of the recommended wall would be 207 cubic yards based off of the area dimensions of the battered wall and the 465 ft length of the wall. This would total

$207,000 for the wall, in addition to the cost of the associated costs with the footing and backfill as described in the precast alternative. The footing and backfill came to be approximately 2/3 of the relative cost for the precast alternative which would be $430,000. These two costs combined come to be $640,000 and with a 15% contingency built in, comes to a total of $736,000.

Leave As-Is

Depending upon other alternatives considered regarding the existing structure and accessibility of Wood Island, was the option to leave the seawall as it is. This option is being considered because rehabilitating the seawall would serve minimal purpose unless the Life Saving Station itself were to be renovated and the seawall would be used as protection for the renovated structure. Other situations, as discussed in the decision matrix, would deem leaving the existing seawall as it is as the most logical solution and are based upon the choices for accessibility and renovation of the existing structure.

Rip Rap Seawall Rehabilitation

The option for placing rip rap in areas where the existing structure needs stability was not researched due to permitting issues which would lie in impacting areas outside the existing seawall. For this reason it was not further looked into, but does heed some recognition as a possible rehabilitation option due to the ease of construction and the relative low cost in comparison to a cast in place or precast seawall.

Rip rap seawalls are comprised of varying sizes of stones which are placed in a way to dissipate wave energy and thus protect the structures behind it from storm damage. A rip rap wall could be placed in areas where the existing seawall is damaged and be used to support the existing wall or act in its place where the existing wall is no longer present.

Seawall Demolition

Depending on which accessibility and Life Saving Station renovation alternatives are chosen, the demolition of the existing seawall may be done to minimize the risk the deteriorating structure currently poses to island visitors.

Construction Logistics

To demolish the existing seawalls a grinder could be used on an amphibious barge and brought onto the island. The grinder consists of a head unit which has teeth which pulverize the existing wall which then uses a conveyor to deposit the material away from the wall. This would serve to eliminate the hazard posed by the wall, and could serve as gravel for pathways along the island or around the Life Saving Station. The option to remove the crushed material from the island was not pursued due to elevated costs associated with doing so. To complete the full demolition of the wall, it would take approximately five work days and would require a three man crew. One person would be to operate the crusher while two laborers would assist the operator as well as use a torch to remove the tiebacks as they proceed as to not impact damage on the teeth of the grinder.

Cost

The conceptual cost associated with the demolition of the existing seawall would be about $1000 for mobilization and demobilization efforts for the grinder, and an additional $3000 for rental of the grinder and the associated crew. Using the five work days needed and adding the mobilization and demobilization costs it would cost approximately $16,000 total.

Recommendations

The seawall is crucial to protecting the Life Saving Structure. For this reason it is recommended to remove the current seawalls and replace them by either constructing a new wall or using one of the precast solutions. The sizes of the walls seem to have been appropriately built, but the walls themselves are in need of new designs. Once new walls are in place the structure should be relatively protected from the ocean's storms.

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