Structural Repair and Protection Cost Estimate and Attendant Condition Appraisal of the Miami Marine Stadium

Miami, FL
January 2010

SGH Project 090457

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
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Project 090457.00 – Structural Repair and Protection Cost Estimate and Attendant Condition Appraisal of the Miami Marine Stadium, Miami, FL

Dear Mr. Worth:

At your request, we conducted a condition appraisal of the Miami Marine Stadium and prepared cost estimates for the structural repair and protection of the stadium structure. Our findings and conclusions are contained in the attached report.

Sincerely yours,

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Letter of Transmittal

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1. INTRODUCTION

At your request, we conducted a condition appraisal of the Miami Marine Stadium (Photo 1) and prepared cost estimates for structural repairs and protection of the stadium structure.

We were assisted in the field investigation and in the preparation of cost estimates by Structural Preservation Systems (SPS) of Pompano Beach, Florida.

2. BACKGROUND AND OBJECTIVE

The Miami Marine Stadium (the stadium) in the Virginia Key area of Miami, Florida is a 6,500-seat, cast-in-place reinforced concrete structure built in 1964. The structure consists of five primary structural systems:

- Foundations: consisting of piles with cast-in-place concrete pile caps supporting columns, grade beams, and a seawall.
- Ground level structure: consisting of grade beams and structural slabs cast on-grade.
- Mezzanine level structure: consisting of slabs and pan-joists supported by beams and columns. At some areas these slabs are cantilevered or supported by hangers connected to the grandstand structure.
- Grandstand structure: consisting of vomitory and parapet walls, and raker and tie beams and columns supporting the tread-and-riser, seating slabs. The treads (seating rows) have a width of 2 ft- 8 in. The risers which support the seating are typically 1 ft high.
- Roof structure: consisting of four hyperbolic paraboloid shells (hypar shells) joined monolithically along a centerline to form a “V”-shaped cross section. Each shell is supported by three inclined (non-vertical) columns, two at the back and one at the interior.

In 1993 in the aftermath of Hurricane Andrew we conducted a condition assessment of the stadium on behalf of others. The main objective of that work was to determine whether the roof structure had been damaged by the hurricane, whether it was safe, and whether hurricane damage, if any, could be repaired. At that time we also conducted a limited condition survey of the stadium structure. The structure has been closed to public use and essentially abandoned since 1992 after Hurricane Andrew, with minimal maintenance or repair efforts since that date.

In 2008 the City of Miami commissioned a study to determine the cost to fully renovate the facility. This study estimated the cost of structural repairs alone to be between $5,000,000 and $15,000,000.

Our primary objective is to better define the cost of the structural repairs and to identify alternatives for protecting the repaired structure to extend its useful life.

3. SCOPE OF WORK

Our findings and recommendations are based on the following scope of work:

1. Document Review: Review of the original construction drawings, and the results of previous condition appraisals of the stadium.

2. Field Investigation: Survey of the condition of selected, representative portions of foundations, ground level, mezzanine level, grandstand and roof structures, and extraction of concrete core samples for laboratory testing.

3. Laboratory Work: Visual and petrographic (microscopic) examination of core samples that we deem to be most representative of the important conditions observed in the visual examinations. Measurement of the chloride content and depth of carbonation of the core samples.

4. Conceptual Design of Remedial Work: Development of conceptual structural repair details to address existing distress and deterioration. Preparation of conceptual structural repair drawings. Identification and analysis of alternatives for protecting the repaired structure to extend its useful life.

5. Cost Estimate: Estimate of the quantities of the various structural repairs. Estimate of the cost of structural repairs and protection.

4. FIELD INVESTIGATION

Gustavo Tumialan, Liying Jiang and Derek Iske visited the stadium from 28 September 2009 through 2 October 2009 to survey the condition of the stadium structure, to conduct GPR surveys, and to extract concrete core samples. Michael Brainerd visited the stadium on 29 September 2009 to make a walk-through inspection of the stadium and to meet with Hilario Candela, the architect who designed the stadium, Jorge Hernandez, member of the Friends of the Miami Marine Stadium, and Colin Meneely and Robert Cunningham of Structural Preservation Systems.

4.1 Condition Survey

As described in our proposal to you, the budget for this work did not allow for a detailed survey of the entire structure. Thus, we utilized the following general methodology for each of the five major structural systems (foundations, ground level structure, mezzanine level structure, grandstand structure, roof structure) to make our selected, representative condition survey meaningful in identifying and estimating the types, extent, and quantities of structural repairs:

- We conducted a brief, cursory visual survey of the entire structure to assess and record which areas or bays were in the best, the worst, and in typical condition.
- We then conducted a more detailed hands-on survey of portions of the structure that we selected from the initial cursory visual survey. Our selections of areas for the detailed survey varied for each of the five major structural systems, but generally, we attempted to select at least one area that represented the typical condition, and one area that represented the worst condition for each major structural system. The specific areas selected for detailed surveys, and a summary of our findings, are presented in drawings SR-1 through SR-4. Our goal in the selection of these areas, and in taking notes during the cursory visual survey of the whole structure, was to provide the basis by which we could estimate and extrapolate repair quantities based on the limited areas of the detailed survey.

In the areas of the detailed surveys, we surveyed the condition of the various structural elements to document the approximate location and extent of distress and deterioration. Our survey included all surfaces of all elements. The survey involved visual observations and sounding with chains or hammers to identify hollow sounding areas (delaminations). Where necessary we used a manlift or a boat for hands-on access. Our general observations are summarized below.

4.1.1 Foundations

- Several piles below the lower seating area show moderate to severe deterioration (Photos 2 and 3). There is a pile showing severe deterioration below the water level (Photo 4).

- At our direction SPS attempted to excavate and dewater an inspection pit at the back of one of the pile caps supporting the main columns that support one of the roof segments. Our intent was to examine and sample one or more piles that are not otherwise exposed to view. It was not possible to accomplish the dewatering using conventional heavy-duty submersible pumps, and consequently, we had to abandon this inspection and sampling.

4.1.2 Ground level structure

- Some columns and beams at the exposed areas between Line 1 and 2 and Lines 16 and 17 show severe deterioration (Photos 5 and 6).

- The seawall along Line D shows areas of severe deterioration (Photos 7 and 8). The concrete cores revealed deeper deterioration which was not detectable using sounding techniques (Photo 9).

- The top sides of the ground level slabs show minor deterioration (Photo 10). At two locations we observed through openings in the slab, the underside side of the slabs and the grade beams are free of distress and deterioration (Photo 11).

4.1.3 Mezzanine level structure

- The steel hangers supporting the mezzanine slab and adjacent concrete show severe deterioration (Photo 12).

- The ramp and mezzanine slabs show moderate deterioration with localized spalls or delaminations (Photos 13 and 14)

4.1.4 Grandstand structure

- The over-water concrete elements (slabs, beams and columns) between Line D and E, and beams north of Line E show severe deterioration (Photos 15 and 16).

- The topside and underside of the seating areas show moderate deterioration with localized spalls or delaminations (Photos 17 and 18).

- Some seating slabs have cracks above the raker beams (Photo 19). The crack widths we measured range between 0.011 in. and 0.020 in. The majority of the cracks are about 0.016 in.

- The slab north of line J has also cracks on the topside. The crack widths range between 0.011 in. and 0.050 in. The majority of the cracks are about 0.020 in.

- The grout plugs at some seating anchor locations are missing or deteriorated due to anchor corrosion (Photo 20).

- The vomitory and parapet walls in general show moderate deterioration with localized spalls or delaminations (Photos 21 and 22). Several parapets around the slant front columns show severe deterioration (Photo 23).
- The raker beams supporting the seating structure show moderate deterioration with localized spalls or delaminations (Photo 24). The beams next to the uppermost seating row shows severe deterioration localized at the beam upper region (Photo 25).

4.1.5 Roof structure

- The inclined columns show moderate deterioration with localized spalls or delaminations (Photos 26 and 27).
- The topside and underside of the roof structure shows moderate deterioration with localized spalls or delaminations (Photos 28 and 29).
- The roof shows diagonal cracking throughout the front cantilever portion (Photos 30 and 31). The crack widths we measured range between 0.013 in. and 0.040 in. The majority of the cracks are about 0.020 in.
- Some post-tensioning anchorage zones of the roof diaphragm are spalled or delaminated (Photo 32).

4.2 Comparisons with 1993 Condition Survey

We retrieved photographs and field notes from our archives to compare the condition of selected structural elements in 1993 with their current conditions. Our comparisons are summarized below.

- The majority of the concrete elements that we compared show spalled areas that have extended since 1993. However, the growth is not alarming. Photos 33 through 38 illustrate comparisons of the conditions of slabs, beams columns and piles.
- There is a tie beam at Line 3 and beam along Line D that shows large spalled areas which were not observed in 1993 (Photo 39).
- The amount and extent of cracking on the roof does not appear to have increased significantly (Photo 40).
- The spalled concrete at the post-tensioning anchorage zone of the diaphragm end at Line 1 does not appear to have increased significantly since 1993 (Photo 41).

4.3 Measurements of Cover over Reinforcement

We measured the concrete cover over reinforcement (cover) at selected concrete elements using ground penetrating radar (GPR) and physical measurement at spalls.

Our limited GPR measurements are summarized below.

- Ground Level Slab – Cover of the top reinforcement on the ground level slab ranges from 1.6 in. to 5 in. with an average of 3.0 in.

- Seawall – Cover of the outermost layer of the reinforcement in the seawall ranges from 2.0 in. to 4.2 in. with an average of 3.5 in.

Our limited physical measurements of concrete cover are summarized below.

- Piles – Cover of the longitudinal reinforcement in the piles ranges from about 1.5 in to 3 in.

- Mezzanine Slab – Cover of the top reinforcement in the mezzanine slab ranges from 5/8 in. to 1 in.

- Seating Slabs – Cover of the reinforcement in the seating slabs ranges from 1/8 in. to 3/8 in.

4.4 Core Sampling

At our direction the assisting Contractor, SPS, extracted concrete 23 core samples from the various structural elements. The locations of the cores are shown on drawings SR-1, SR-2, SR-3, and SR-4.

5. LABORATORY WORK

5.1 Chloride Content

We tested the chloride content of the concrete in core samples in accordance with AASHTO T-260 Acid Soluble Method. We also compared the results of the tests performed in 1993 and 2009 on concrete samples extracted from the roof hypars. Our laboratory report of the chloride content tests is contained in Appendix A. The chloride content test results are summarized in the following tables.

Table 1 – Results of Chloride Content Testing

Sample ID	Element	Location	Depth From Surface	Chloride Ion	
			(in.)	%	lbs/yd^3
C1	Slab-on-grade - between column lines 3 and 4	Top	1/4	0.163	6.38
		Middle	3-1/4	0.041	1.61
		Bottom	6	0.022	0.86
C3	Mezzanine Level - Cantilever slab - between column lines 5 and 6	Top	1/4	0.129	5.05
		Middle	1-1/2	0.007	0.27
		Bottom	7-3/4	0.078	3.05
C6	Lower Seating - Row 2 trend - between column lines 2 and 3	Top	1/4	0.168	6.58
		Middle	8	0.008	0.31
		Bottom	15-3/4	0.169	6.62
C9	Upper Seating - Row 41 trend - between column lines 2 to 3	Top	1/4	0.102	3.99
		Middle	1-3/4	0.021	0.82
		Bottom	3/1/4	0.022	0.86
C19	Upper Seating - Beam/Wall above Row 41 - at column line 6	Exterior	1	0.132	5.17
			3	0.064	2.51
C13	Pile F-6 at tide exposed zone	Exterior	1	0.346	13.55
			2	0.366	14.33
C14	Pile F-14 at tide exposed zone	Exterior	1	0.728	28.5
			2	0.719	28.15
C18	Pile F-14 18" above tide exposed zone	Exterior	1	0.877	34.33
			2	1.121	43.89
C11	Seawall - between column lines 5 and 6	Exterior	2	0.312	12.21
			3-1/2	0.638	24.98
			5-1/2	0.223	8.73
C15	Seawall - between column lines 14 and 15	Exterior	2	0.644	25.21
			3-1/2	0.597	23.37
C17	Vomitory Wall – column line 13	Exterior	1/4	0.153	5.99
		Middle	3-3/4	0.022	0.86
		Interior	7	0.062	2.43
C8	Low portion of tension column at column line 2	Exterior	1/2	0.032	1.25
			1/1/2	0.014	0.55
C23	Upper portion of tension column at column line 3	Exterior	1/2	0.032	1.25
			1-1/2	0.029	1.14
C20	Roof - Front Hypar - between column lines 7 and 8 - Near 1993 core C5	Top	1/4	0.079	3.09
		Middle	1-1/2	0.035	1.37
		Bottom	3	0.04	1.57
C22	Roof - Front Hypar - between column lines 12 and 13 - Near 1993 core C6	Top	1/4	0.043	1.68
		Middle	1-1/2	0.031	1.21
		Bottom	3	0.06	2.35
C21	Roof- Diaphragm - between column lines 9 and 10	Exterior	1	0.027	1.06
			2-1/2	0.013	0.51

Table 2 – Comparison of Results of Chloride Content Testing

Sample ID	Location	Depth From Surface	Chloride Ion 2009		Chloride Ion 1993	
		(in.)	%	lbs/yd^3	%	lbs/yd^3
C20	Top	1/4	0.079	3.09	0.022	0.90
	Middle	1-1/2	0.035	1.37	0.01	0.40
	Bottom	3	0.04	1.57	0.052	2.00
C22	Top	1/4	0.043	1.68	0.022	0.90
	Middle	1-1/2	0.031	1.21	0.013	0.50
	Bottom	3	0.06	2.35	0.043	1.70

5.2 Carbonation Testing

We conducted tests of the depth of carbonation in core samples by applying phenolphthalein to freshly cut concrete surfaces. Our laboratory report of the carbonation tests is contained in Appendix A. The carbonation test results are summarized in the following table.

Table 3 – Results of Carbonation Testing

Sample ID	Element	Top Down		Bottom Up	
		Min. (in.)	Max. (in.)	Min. (in.)	Max. (in.)
C1	Slab-on-grade - between column lines 3 and 4	0	5/16	None	None
C3	Mezzanine Level - Cantilever slab - between column lines 5 and 6	1/16	3/8	None	None
C6	Lower Seating - Row 2 trend - between column lines 2 and 3	None	None	0	1/4
C9	Upper Seating - Row 41 trend - between column lines 2 to 3	1/8	3/16	0	1/8
C19	Upper Seating - Beam/Wall above Row 41 - at column line 6	0	9/16	None	None
C13	Pile F-6 at tide exposed zone	None	None	None	None
C14	Pile F-14 at tide exposed zone	None	None	None	None
C18	Pile F-14 18 in. above tide exposed zone	None	None	None	None
C11	Seawall - between column lines 5 and 6	None	None	None	None
C15	Seawall - between column lines 14 and 15	None	None	None	None
C17	Vomitory Wall – column line 13	None	None	0	1/8
C8	Lower portion of back inclined column at column line 2	0	1/2	None	None
C23	Upper portion of back inclined column at column line 3	0	1/16	None	None
C20	Roof - Front Hypar - between column lines 7 and 8 - Near 1993 core C5	0	5/16	0	11/16
C22	Roof - Front Hypar - between column lines 12 and 13 - Near 1993 core C6	0	0	0	1/32
C21	Roof- Diaphragm - between column lines 9 and 10	None	None	None	None

5.3 Visual and Petrographic Examination

We cut, polished and examined Cores C8 (inclined column), C9 (upper seating), C11 (seawall), C13 (pile, at tide-exposed zone), C14 (pile, at tide-exposed zone), C18 (pile, above tide-exposed-zone), and C19 (upper seating beam) to identify the composition of concrete and material-related deterioration. We examined the polished sections with the aid of a stereomicroscope at magnifications of 6 to 50X. In addition, we prepared blue-dye epoxy-impregnated, ultrathin sections from representative portions of Cores C11 and C13 to conduct a more-detailed petrographic examination and evaluation of the composition and overall quality of the hardened concrete. We examined these sections using a transmitted-light polarizing microscope at magnifications of 25 to 200X. The following is a summary of our observations:

- The overall quality of the concrete in all seven core samples appears to be fair to good, with well graded and uniformly distributed aggregates that exhibit a moderately well to well developed paste-to-aggregate (P/A) bond strength as determined by the frequency of fractured aggregate particles on fresh fracture surfaces of the hardened concrete.

- There are no gross differences in the materials or apparent mix proportions between the seven core samples, with the exception of differences in the apparent water-to-cementitious-material (w/cm) ratio of the concrete between individual core samples.

- The majority of the hardened concrete is air entrained. We estimate that the total air content of the hardened concrete ranges from 2.5% to 3.5%, as determined by a comparison with known laboratory standards, with slightly differences between individual core samples.

- There is a 3/8 to 1/4 in. thick layer of a dark colored coating material on the exterior surface of the concrete in Core C11. The coating appears to be a sand-filled polymer waterproofing.

- The concrete in Core C11 exhibits characteristics associated with a highly variable w/cm that we estimate varies from low to moderate (0.40 to 0.42) to moderate and moderately high (0.48 to 0.54) within the body of the core sample. This variation in w/cm ratio indicates that the concrete was either retempered or was inadequately mixed prior to being discharged. By comparison, the concrete in Core C13 exhibits more uniform w/cm that we estimate to be in the range of 0.46 to 0.52. Overall, the w/cm estimates for the hardened concrete in the remaining five core samples ranges from 0.40 to 0.52.

- The near surface concrete exhibits variable carbonation depths among the seven core samples, which is attributable to variations in the initial w/cm and differences in the exposure conditions. The carbonation depths are as shallow as 0 in. to 1/8 in. (Cores C9, C14 and C18) and as deep as 11/16 in. (Cores C20).

- There is evidence of severe corrosion and formation of corrosion deposits on existing fracture surfaces in Cores C11 and C14, and in the body of Core C18. In the case of Core C18, there is a corrosion stained horizontal crack that fractures the concrete at 2-1/4 in. below the exterior surface. In addition, there are a great number of white salt deposits on the prepared polished surface in Core C18 (pile). The white salt deposits suggest that the concrete in Core C18 was exposed to an environment with significant amounts of chloride that penetrated deeply into the body of the concrete, which in turn caused the corrosion of reinforcing steel except for occasional early age drying shrinkage cracks. There is no cracking in the majority of the body of the concrete in the examined core samples. However, there are occasional near horizontal crack in the concrete that are attributed to corrosion of the reinforcement (C18). In Core C19, there are multiple fine cracks that extend from the surface of the core into the body of the concrete. The path, orientation, and location of these fractures indicate that during the construction, the concrete was disturbed prior to achieving final set, which caused the fractures to form while the concrete was in a semi-plastic state.

- Other than variations in carbonation depths and chloride-induced corrosion of the steel reinforcement, there is no other evidence of other forms of deterioration or chemical alteration in the hardened concrete, such as alkali-aggregate reactivity, or sulfate or acid attack.

6. DISCUSSION

6.1 Background on General Mechanisms of Concrete Deterioration

Concrete normally provides excellent protection against the corrosion of embedded steel reinforcement – the concrete shields the steel reinforcement from direct exposure to water and the elements, and the alkaline environment of the concrete further protects the steel from corrosion. However, over time, several factors including cracking, chlorides, and carbonation can break-down this protection. Cracking (for example, shrinkage cracks) can allow moisture direct access to the steel reinforcement. The natural process of carbonation (the reaction of the concrete with carbon dioxide and humidity in the air) causes the naturally high alkalinity of the concrete to decrease, starting at the outer face and proceeding inward. In some cases chlorides are present in the original concrete mix (for example, if sea sand was used, or if salt was added to the mix). In some cases the chloride level of the concrete can increase through exposure to salt spray or salt water in marine environments.

Chlorides and carbonation can break-down the natural protection of the concrete against corrosion of the steel reinforcement. The depth of chloride and carbonation penetration increases with time. When the amount of chloride at the depth of the reinforcement reaches about 1.25 lbs/cy, or when the carbonation front reaches the reinforcement, corrosion will occur in the presence of moisture and oxygen. The corrosion produces rust that expands and causes delamination and spalling of the concrete over the reinforcement. Cracking and spalling opens greater and deeper avenues for the ingress of water, the carbonation of the concrete, and ingress of environmentally deposited chlorides (like sea water or salt spray), consequently, with any of these causes, the rate of deterioration of the concrete tends to increase over time.

Typically, the time-to-corrosion of the steel depends on a number of factors including the permeability of the concrete (which is proportional to the water/cement ratio of the concrete), the depth of cover, the extent of cracking, and the conditions and severity of the exposure. The higher the permeability, the thinner the cover, the harsher the exposure, and the more cracking, the sooner corrosion of the steel reinforcement and delamination of the concrete will begin to occur. Marine environments, in particular, are one of the most severe and aggressive environments for concrete due to the high chloride concentration in seawater, salt-laden rains, and wetting and drying cycles due to tidal changes and wave splashing. Seawater and precipitation in marine environments contain salt (a chloride) that permeates exposed concrete surfaces. Cracks allow seawater and salt laden-water to permeate the concrete more quickly. Marine structures are exposed to chlorides from seawater in four exposure conditions:

- Submerged Zone: This zone includes the structural members or part of structural members that are submerged all times. In the stadium the piles regions below the mean low water level are in the submerged zone
- Tidal Zone: This zone includes the structural members that are alternately exposed and submerged due to tidal fluctuation. In the stadium the pile regions between the mean low and mean high water are in the tidal zone.
- Splash Zone: This zone includes the structural members that are affected by the action of waves. In the stadium, the pile regions above the mean high water level, seawall, and structural elements below the lower seating are in the splash zone.
- Open Zone: This zone includes the structural members that are mainly exposed to sprayed seawater, salt-laden rain and moisture. In the stadium, the upper and lower seating structure, ground and mezzanine slabs, ramps, inclined columns and roof structure are in the open zone.

6.2 Condition of the Miami Marine Stadium Structure

The stadium has been exposed to the marine environment, and the salts (chlorides) inherent in that environment) for about forty-five years. This severe exposure has caused corrosion of the reinforcement and subsequent delamination and spalling of many concrete elements.

The present deterioration of the concrete does not appear to be due to inadequate quality of the concrete, nor carbonation, nor alkali-silica reaction of the aggregate, nor acid or chemical attack of the concrete. Rather, the high chloride content in the concrete appears to be both the major contributor to the present concrete deterioration, and the greatest threat and obstacle to the future durability of the stadium's concrete structure. The varying levels of chlorides at different depths within individual concrete samples (cores), and the higher levels of chlorides in the areas of the stadium with a more severe exposure to salt water or salt spray, indicates that the high chloride levels in the concrete are from exposure to the marine environment over time, and not from chlorides in the original concrete mix.

The various concrete elements throughout the stadium show different degrees of deterioration ranging from moderate to severe. The majority of the severe deterioration occurs on the concrete elements located in the tidal and splash zones. Some of these elements require replacement due to the severity and extent of deterioration. The structural elements in the open zone show moderate deterioration for the most part. When compared to the conditions found in 1993, the amount of deterioration does not appear to have increased alarmingly. However, since these observations are limited to the elements or areas that we were able to identify in 1993 photographs or notes, we cannot generalize the observed deterioration rate for the balance of the structural elements in the stadium.

Our petrographic analyses indicate that the concrete has a w/cm ratio estimated to vary from 0.40 to 0.54, indicating that the concrete has low to moderate permeability. The maximum depth of carbonation we measured is about 0.70 in. in a concrete roof sample. Most of the carbonation measurements are less than 0.50 in. The rule-of-thumb carbonation rate for average quality concrete is about 1 mm (0.04 in.) per year for the first three years and 0.50 mm (0.02 in.) per year thereafter. At this rate, the carbonation after forty-five years would be about 1 in. The apparent relatively low carbonation rate of the concrete is likely due to the permeability of the concrete.

Concrete in marine environments are prone to corrosion due to external chlorides found in saltwater and salt-laden rain. The vast majority of the cores show chloride contents that are in excess the threshold of 1.25 lbs/cy for corrosion. The chlorides appear to be significantly increasing with time. This is evident from comparisons that we made on chloride test results conducted in 1993 and 2009 on samples extracted from the roof.

The chloride content in tidal-zone and splash-zone elements such as piles and seawall is extremely high. In these elements the chloride content at the level of the reinforcement is up to 20 times the threshold. In open-zone elements, the chloride content is also very high. In some elements such as the lower seating and the ground and mezzanine level slabs the chloride threshold is about 5 times the threshold.

The chloride tests on samples extracted from the lower and upper seating revealed that the chloride content in the lower seating is about 1.6 times higher than in the upper seating. This difference may be attributed to the roof structure which provides more cover to the upper seating. Also, during our site visit we observed a fireboat pumping water into the stands. The water streams mainly wetted the lower seating, reaching up to the first rows of the upper seating. The water reached the mezzanine and ground slabs through the vomitories. We do not know the purpose and frequency of these operations but they can certainly lead to increases in chloride content.

The concrete cover we measured on piles at tidal-zone and splash-zone elements such as piles and seawall ranges from 1.5 in. to 3 in, and from 2.0 in to 4.2 in., respectively. The concrete cover we measured in open-zone elements is variable. For instance, in the seating slabs, the cover ranges from 1/8 in. to 3/8 in., whereas in the topside of the ground level slab the cover ranges from 1.6 in. to 5 in. The original drawings specify concrete covers of 2 in. for columns and beams, and 1 in. cover for slabs. Comparison of the concrete covers (measured and specified) with the chloride contents suggests that corrosion damage will continue to occur even after corrosion damage is repaired.

Although we examined all of the exposed-to-view portions of the seafront piles, the vast majority of the piles supporting the structure are not exposed-to-view. Our attempt to access, inspect and sample hidden piles was unsuccessful. The piles have spent over 45 years in a seawater environment and their condition is largely unknown. It would be prudent to make some further assessment of the piles. Such an assessment could include one or more of the following tasks:

- Researching the performance of similar pile systems in similar environments.
- Inspecting and sampling some below-grade piles at inspection pits accomplished with more robust dewatering, which could include installation of cofferdams with tremie seals, if necessary, to cutoff water inflow to the inspection area.
- Making an elevation survey of the structure at the locations of the foundations to look for signs of settlement.
- Non-destructive testing of piles using techniques such as sonic-echo testing and/or parallel seismic testing to allow the length and integrity of the pile to be investigated.

Such investigations, especially the inspection and testing of hidden piles, are very costly. Consequently, accomplishing a statistically meaningful assessment would be prohibitively expensive. It may come to pass that the some or all of pile foundations require remediation before the end of the useful life of the balance of the rehabilitated stadium structure. Further pile assessment would provide at least some indication of the likelihood of future need for pile foundation remediation.

7. REMEDIAL WORK

Remedial work to the stadium's concrete structure generally falls into two categories:

- Concrete Repairs: These are necessary to repair existing damage to the concrete structure.

- Corrosion Mitigation Measures: These measures do not address existing damage, and are not necessary to make the structure safe and serviceable in the immediate future. Some level of corrosion mitigation, however, is often prudent in conjunction with the concrete repairs to extend the useful life of the structure, by slowing the future rate deterioration.

These general classifications of remedial work to the concrete structure are further discussed below.

7.1 Concrete Repairs

Based on our condition appraisal of the stadium as set forth herein, we identified the concrete repair necessary to repair existing deterioration and of the stadium structure.

The delaminated and spalled concrete can be repaired using conventional concrete repair techniques, involving removal of deteriorated concrete, removal of concrete from around exposed reinforcing bars, application of anti-corrosion inhibitors on the concrete surface, application of protective coating on the reinforcement, installation or sacrificial anodes at repair cavities, and filling the repair cavities with a repair material.

Structural members showing extensive deterioration should be replaced with new cast-in-place concrete members. This work will involve demolition of the deteriorated member, installation of new epoxy-coated reinforcement, forming and casting of new member. Shoring will be required during replacement of columns to support the existing structure during demolition and repair.

The concrete mixture for the new cast-in-place concrete elements should be designed to minimize chloride attack. Specifically, the water-cementitious ratio should not be larger than 0.40 and the mix should have a corrosion inhibitor admixture. A low w/cm ratio will make the concrete less permeable to chlorides, and the corrosion inhibitor will provide further protection as the chlorides penetrate the concrete, albeit more slowly.

Drawings SR-1 through SR-6 show the estimated extent of the repairs, approximate locations and conceptual repair details. These conceptual details are adequate for this preliminary cost estimating, but they are not suitable for construction. Prior to construction, during the design development and construction document preparation phases, the repair quantities need to be updated, and the repair details, materials, and execution requirements need to be more completely defined. Below is a general summary of the required structural repairs.

7.1.1 Foundations

- Repair delaminated and spalled concrete on exposed piles.
- Install new piles to supplement deteriorated exposed piles.

7.1.2 Ground Level Structure

- Repair delaminated and spalled concrete on the seawall.
- Repair delaminated and spalled concrete on the topside of the slabs.
- Repair delaminated and spalled concrete on columns.
- Remove and replace severely deteriorated beams, slabs and columns.

7.1.3 Mezzanine Level Structure

- Repair delaminated and spalled concrete on the topside, underside and edges of the slabs.
- Replace deteriorated steel hangers and connectors supporting the mezzanine slab.

7.1.4 Grandstand Structure

- Repair delaminated and spalled concrete on the topside and underside of the seating slabs.
- Repair delaminated and spalled concrete on beams.
- Remove and replace severely deteriorated beams, slabs and columns.
- Repair delaminated and spalled concrete on vomitory and parapet walls, and roof diaphragm.
- Fill cracks wider than 0.012 in. in grandstands beams by epoxy injection.
- Replace delaminated parging on walls and columns.
- Install sealant in existing seating anchor pockets with missing grout plugs. Remove debonded or cracked grout and install sealant.

7.1.5 Roof Structure

- Repair delaminated and spalled concrete on the topside and underside of the roof and wall diaphragm.
- Fill cracks in the roof by epoxy injection.

7.2 Corrosion Mitigation Systems

Corrosion of the reinforcement will continue to occur at locations where the concrete is chloride-contaminated to the depth of the reinforcement. Fortunately, there are corrosion mitigation technologies that can be used individually or in combinations to slow, and in some instances theoretically stop ongoing corrosion damage. Based on their application methods, these technologies can be divided in external and internal protection systems. External systems include penetrating sealers, liquid applied waterproofing, and anti-carbonation coatings. Internal systems include protective coatings for exposed reinforcement, corrosion inhibitors, cathodic protection, galvanic protection, chloride extraction, and re-alkalinization. These corrosion mitigation systems are described below.

External Corrosion Mitigation Systems: Generally, these systems tend to be surface-applied fluids that slow the rate of corrosion by reducing the ingress of moisture and/or air into the concrete. These measures tend not to reverse any existing internal conditions within the concrete that may be promoting deterioration, such as high chloride content, or loss of alkalinity as a result of deep carbonation.

- **Clear, Penetrating Water-repellent Sealer:** Currently, there are many sealers designed to reduce the ingress of moisture into exposed concrete and thereby extend the useful life of the concrete and concrete repairs by slowing, but not stopping, corrosion activity. Penetrating sealers are not a complete barrier, do not bridge cracks, and do not slow the advancement of carbonation. Penetrating sealer must be reapplied periodically, usually about every five to seven years. Penetrating sealers are available in a clear form that would not alter the historic appearance of the exposed cast-in-place concrete.
- **Liquid-Applied Waterproofing:** Liquid-applied waterproofings are opaque waterproofing coatings, typically heavy-bodied, designed to prevent the ingress of water at critical surfaces such as roofs or decks. They can include various material classes, often, elastomerics, acrylics, urethanes, epoxies, etc. They are often applied in multiple coats. They can also include aggregates broadcast into the topcoat to improve traction and slip-resistance on walkable or driveable surfaces such as decks and slabs. Liquid applied waterproofing coatings, tend to need to be re-applied every 5-15 years, depending on exposure to wear. These coatings are opaque, and have the appearance of a very thick paint or coating, thus they significantly alter the historic

appearance of the exposed cast-in-place concrete that was not painted or coated originally.

- **Elastomeric or Anti-carbonation Coatings:** Currently, there are many coatings designed to protect exposed concrete from the elements (moisture, chloride, carbonation, acid rain, etc.) and thereby extend the useful life of the concrete and concrete repairs by slowing, but not stopping, corrosion activity. Bug holes in the concrete must be filled prior to application of the coating to prevent defects in the coating. This will involve applying a parge coat of filler material prior to application of the coating. To a certain extent, any film-forming coating, even a conventional acrylic household paint, retards the rate of air and moisture ingress into the substrate concrete, and thus reduces the rate of carbonation. Thus, to a certain extent, the term "anti-carbonation coating" may represent clever marketing more than it represents any truly significant technical breakthrough. Anti-carbonation coating must be re-applied periodically, usually about every 10 to 15 years. These coatings are opaque, and have the appearance of a paint or coating, thus they significantly alter the historic appearance of the exposed cast-in-place concrete that was not painted or coated originally.

Internal Corrosion Mitigation Systems: Generally, these treatments tend to work at the sub-surface level (internal to the concrete) by either chemically altering the concrete or reinforcing steel, or by electrically changing the internal micro-climate within the concrete to slow the rate of corrosion of the internal steel reinforcement. In this way, some of these measures are capable of addressing and reversing internal root causes of concrete deterioration (e.g., chloride extraction can remove internal chlorides, realkalization can restore lost internal alkalinity).

- **Migratory Corrosion Inhibitor:** Migratory inhibitors are chemicals applied to the concrete surfaces to saturation. These materials theoretically migrate through the concrete to the embedded steel. Once at the level of the steel, the inhibitors interfere with the corrosion process and slow the rate of corrosion. Corrosion inhibitors might require reapplication depending on the severity of exposure. Migrating corrosion inhibitors are clear, penetrating fluid application that will not alter the historic appearance of the exposed cast-in-place concrete.

- **Galvanic Protection:** Premature corrosion of reinforcement at the perimeter of repairs often occurs due to potentials created at the boundary between the new, uncarbonated repair material and the original carbonated concrete. Zinc-based composites embedded in the repairs can prevent such premature "incipient anode" corrosion. The zinc sacrifices itself to protect the steel just outside the repair and prevents incipient anodes from establishing. Sacrificial anodes in repairs do not provide protection to the entire structure. However, sacrificial anodes placed in holes drilled away from repairs can be used to protect the steel away from repair areas. Zinc composites have a finite life since they are consumed as they protect the steel. Typical life expectancy is about 20 years. As the embedded zinc composites are entirely internal, they will not alter the historic appearance of the exposed cast-in-place concrete when added at repair locations.

- **Cathodic Protection:** Cathodic protection, which controls the corrosion of the reinforcing steel with impressed current, can in theory stop corrosion of the embedded steel. Current is continually supplied to the reinforcement using a rectifier. The rectifier supplies current to a grid over the element to be protected. The grid is a series of wire-like anodes that are embedded in the structure. The anodes are set into slots that are cut into the concrete or in drilled holes. Conduits and junction boxes are needed to feed current from the rectifier the structural elements and may be visible. Current is very low and power consumption is negligible. The series of slots and holes cut into the concrete surface and then "patched" (repaired) can alter the appearance of the original concrete surface, as will the surface-mounted conduits and junction boxes.

- **Chloride Extraction:** Chlorides can be removed from concrete by the temporary application of an electric field to the reinforcement. This technology requires the installation of a conductive mesh of titanium or steel attached to the concrete surface, which acts as an anode, and a wet cellulose fiber slurry sprayed onto the mesh. The concrete is wrapped in plastic to minimize evaporation. Current is passed through the reinforcing steel, which acts as cathode, and the chloride ions migrate from the reinforcement toward the mesh. The process must be applied and run for several weeks to achieve results. Along with re-alkalization, it is the only process discussed herein which addresses the root cause of the corrosion and the consequent spalling. When the process is complete, the chloride in the concrete is greatly reduced, after which, the titanium mesh is removed. It is the <u>only</u> process discussed herein which addresses the primary root cause of the corrosion of steel reinforcement and the consequent spalling and deterioration of the concrete. Chloride extraction will not alter the historic appearance of the exposed cast-in-place concrete.

- **Re-alkalization:** Concrete naturally is highly alkaline (has a high pH), but it loses this natural alkalinity over time as the concrete reacts with moisture and carbon dioxide in a natural process known as carbonation. The protective alkalinity of the concrete can be restored permanently through temporary application of a highly alkaline fluid or paste, in combination with an electric field to the reinforcement. This involves the application of a titanium mesh as a temporary anode and a fabric to the surface of the concrete. The fabric is kept wet with a special highly alkaline solution, wrapped in plastic, and a current is passed through the reinforcing steel to the titanium mesh anode. The process permanently restores the high pH around the reinforcing steel. The process must be applied and run for several days to achieve results, after which, the titanium mesh and fabric are removed. Re-alkalization will not alter the historic appearance of the exposed cast-in-place concrete.

7.3 Remedial Work Alternatives

The repair of deteriorated (e.g., cracked, spalled, delaminated) concrete is clearly necessary for the rehabilitation and reuse of the structure. The cost of those necessary structural repairs is identified in the cost estimates herein provided by SPS. The final design of the repairs, and further field work to update repair quantities will be necessary at the design stage.

While corrosion mitigation measures are not necessary for restoring the structural integrity of the stadium, some level of corrosion mitigation would be prudent in combination with the concrete repairs, to:

- Protect the initial investment in the concrete repairs.
- Extend the useful life of the structure.
- Reduce future maintenance and repair costs to the structure after the initial major repair/rehabilitation.

While countless permutations exist for logical and viable combinations of various corrosion mitigation treatments in various areas of the stadium for cost estimating purposes, we identified the following four basic alternatives among those that we consider to be technically acceptable to rehabilitate and extend the useful life of the concrete structure. Because carbonation is not a significant cause of deterioration of the concrete, there would be no logical reason to use re-alkalization as a treatment on the Miami Marine Stadium. Each alternative includes the same repairs to the existing areas of deteriorated concrete, along with various combinations corrosion mitigation treatments to reduce the rate of future deterioration, and to reduce the future cost of repairs. They are listed in order of lowest cost and lowest protection to highest costs and highest protection.

- **Alternative 1 – Concrete Repairs and Roof Waterproofing**

This repair alternative involves the following work:

- Repair of deteriorated elements and replacement of severely deteriorated elements.
- Installation of liquid-applied waterproofing on the topside of the roof.

Figure 1 illustrates the members receiving corrosion mitigation systems for this alternative.

- **Alternative 2 – Concrete Repairs, Corrosion Inhibitor and Roof Waterproofing**

This repair alternative involves the following work:

- Repair of deteriorated elements and replacement of severely deteriorated elements.
- Installation of liquid-applied waterproofing on the topside of the roof and diaphragm.

- Application of migrating corrosion inhibitors on the topside and underside of the roof, grandstands, and slabs, and on the exposed surfaces of beams, columns and walls.

Figure 2 illustrates the members receiving corrosion mitigation systems for this alternative.

- **Alternative 3 – Concrete Repairs, Corrosion Inhibitor, Water Repellent, and Roof, Grandstand and Slab Waterproofing**

This repair alternative involves the following work:

- Repair of deteriorated elements and replacement of severely deteriorated elements.
- Installation of liquid-applied waterproofing on the topside of the roof and diaphragm, and liquid-applied pedestrian-traffic waterproofing system on the top sides of grandstands, and slabs.
- Application of migrating corrosion inhibitors on the bottom sides of the roof, grandstands, and slabs, and on the exposed surfaces of beams, columns and walls.
- Application of a clear, penetrating, water-repellant sealer on the bottom sides of the lower seating grandstands and on the exposed surfaces of beams, columns and walls supporting the lower seating.

Figure 3 illustrates the members receiving corrosion mitigation system for this alternative.

- **Alternative 4 – Concrete Repairs, Chloride Extraction, Corrosion Inhibitor, Water Repellant, and Roof, Grandstand and Slab Waterproofing**

This repair alternative involves the following work:

- Repair of deteriorated elements and replacement of severely deteriorated elements.
- Installation of liquid-applied waterproofing on the topside of the roof.
- Extraction of chloride from the grandstands and slabs.
- Application of migrating corrosion inhibitors on the bottom sides of the roof, grandstands, and slabs, and on the exposed surfaces of beams, columns and walls.
- Application of clear, penetrating, water-repellant on the exposed surfaces of beams, columns and walls supporting the lower seating.

Figure 4 illustrates the members receiving corrosion mitigation systems for this alternative.

Other combinations, and the advantages and disadvantages of each, should be further studied at the schematic design phase for the complete stadium rehabilitation. Some general considerations and areas for further study during the design phase include:

- If the migrating corrosion inhibitor would likely be effective on this structure, but would require re-application at a future date, and may be best not to put a penetrating water repellant sealer or a coating on both sides of the slab or structural element in those areas, as that would hinder absorption of the reapplication of the migrating corrosion inhibitor at a later date.

- Applying a penetrating water repellant sealer or a coating on both sides of the slab or structural element may reduce the absorption of salts into the element, but may also inhibit the drying of incidental moisture within the element – thus the advantages and disadvantages of this trade-off, and the vapor permeability (i.e., "breathability") of the underside water repellant would need to be studied and evaluated further.

- Applying a waterproofing or other opaque coating (e.g., elastomeric coating) to concrete that was never painted or coated historically would drastically change its appearance. For this reason, we have not included opaque coatings on previously uncoated surfaces within the four initial options, with the exception of the topside of the grandstand deck and slabs. These areas are particularly vulnerable to corrosion damage, because of the low cover on the concrete. Further, they would benefit greatly from a coating in allowing for routine maintenance such as (fresh) water cleaning of the grandstand seating areas and walking surfaces after concerts and other stadium events, while minimizing water ingress into the slab, where it can promote damage to the near-surface steel. The pros and cons of additional protection and durability over the structure in the most vulnerable areas, along with preservation of the original appearance wherever possible, should be further explored and evaluated during the design phase.

- Because the concrete is chloride contaminated it is likely that corrosion damage will continue to occur even after existing corrosion damage is repaired and corrosion mitigation measures are implemented. The rate of ongoing corrosion damage and the frequency and amount of concrete repair in the future will depend on the effectiveness of the corrosion mitigation measures that are implemented. With external corrosion mitigation measures alone another cycle of significant concrete repairs could be required in as little as 10 to 15 years. With internal corrosion mitigation measures the time to the next cycle of repair might be increased significantly beyond 10 to 15 years. The state of the art of predicting the future performance of rehabilitated corrosion damaged structures is in its infancy; consequently, comparison of the costs and benefits of the various corrosion mitigation measures is imprecise but still worth some level of evaluation.

- Because chlorides are the primary cause of the existing deterioration of the concrete structure, and the greatest threat to the future durability of the structure, chloride extraction is the only treatment that theoretically removes the primary root cause of the concrete deterioration from the concrete. Thus, its use should be strongly considered, even though it is one of the more expensive treatment options.

- The internal corrosion mitigation systems, other than cathodic protection, have relatively short use histories compared to the external systems. Consequently, it might be sensible to include trial installations of one or more of these internal systems over small portions of the structure as part of the stadium rehabilitation rather than implementing them fully throughout at this time. The trial installations would provide valuable information on both constructability and cost. The effectiveness of the trail installations can be monitored over an extended period of time to provide information helpful in deciding whether implementing such an internal protection system would be sensible at some time in the future.

8. CONCEPTUAL-REPAIR COST ESTIMATES

We asked SPS to prepare budget estimates of the costs for the remedial work alternatives described above. SPS is a specialty concrete repair contractor who is familiar both with the local market and construction costs, and who have direct knowledge and experience with the Miami Marine Stadium. For their estimating purposes, we prepared drawings SR-1 through SR-6, showing the approximate locations of repairs, conceptual repair details, and material and execution requirements for repair and protection. We provided SPS with estimated repair quantities based on our visual observations and extrapolation of our detailed surveys. The detailed cost estimates prepared by SPS for each alternative are contained in Appendix B. We added a 15 percent contingency for design and construction. The cost estimates are summarized below.

• Alternative 1 – Concrete Repairs and Roof Waterproofing	**$ 5,660,000**
Concrete Repairs	$ 4,500,000
Corrosion Mitigation	$ 420,000
Contingency (15%)	$ 740,000
• Alternative 2 – Concrete Repairs, Corrosion Inhibitors and Roof Waterproofing	**$ 6,050,000**
Concrete Repairs	$ 4,500,000
Corrosion Mitigation	$ 760,000
Contingency (15%)	$ 790,000
• Alternative 3 – Concrete Repairs, Corrosion Inhibitors, Water Repellent, and Roof, Grandstand and Slab Waterproofing	**$ 6,580,000**
Concrete Repairs	$ 4,500,000
Corrosion Mitigation	$ 1,220,000
Contingency (15%)	$ 860,000
• Alternative 4 – Concrete Repairs, Chloride Extraction, Corrosion Inhibitors, Water Repellant, and Roof, Grandstand and Slab Waterproofing	**$ 8,510,000**
Concrete Repairs	$ 4,500,000
Corrosion Mitigation	$ 2,900,000
Contingency (15%)	$ 1,110,000

The estimates include the contractor's overhead and profit. The estimates excluded engineering fees, construction interest, preparation of mockups and other soft costs. Also, these estimates are solely for the existing concrete structure, and do not include modifications or additions such as replacement of railings, construction of new access ramps, and replacement or repair of the press box.

The actual costs may vary, up or down, from these estimates for many reasons, including, but not limited to, increase in deterioration over time, changes during design development and final design, and the business climate at the time of bidding and construction.

9. CONCLUSIONS

Based on the condition assessment of the Miami Marine Stadium as set forth herein, we find and conclude the following.

1. Repair of the concrete structure for safe public use is technically feasible.

2. In addition to the repair of deteriorated concrete, some combination of corrosion mitigation measures would be prudent to:

   - Protect the initial investment in the concrete repairs
   - Extend the useful life of the structure
   - Reduce future maintenance and repair costs to the structure after the initial major repair/rehabilitation

3. The most suitable combination of corrosion mitigation measures, for different areas of the structure, should be further analyzed and studied during the design phase, but the four alternatives included herein should adequately cover the typical range of costs for likely combinations.

4. Because the concrete is chloride contaminated it is likely that corrosion damage will continue to occur even after existing corrosion damage is repaired and corrosion mitigation measures are implemented. The rate of ongoing corrosion damage and the frequency and amount of concrete repair in the future will depend on the effectiveness of the corrosion mitigation measures that are implemented. With external corrosion mitigation measures alone another cycle of significant concrete repairs could be required in as little as 10 to 15 years. With internal corrosion mitigation measures the time to the next cycle of repair might be increased significantly beyond 10 to 15 years.

5. Estimated costs for repair and rehabilitation for the concrete structure, and additional measures to reduce the future rate of deterioration and future cost of repairs, are as follows:

   - Alternative 1 – Concrete Repairs and Roof Waterproofing only — $ 5,660,000
   - Alternative 2 – Concrete Repairs, Corrosion Inhibitors and Roof Waterproofing — $ 6,050,000
   - Alternative 3 – Concrete Repairs, Corrosion Inhibitors, Water Repellent, and Roof, Grandstand and Slab Waterproofing — $ 6,580,000
   - Alternative 4 – Concrete Repairs, Chloride Extraction, Corrosion Inhibitors, Water Repellant, and Roof, Grandstand and Slab Waterproofing — $ 8,510,000

6. The piles supporting the structure have spent over 45 years in a seawater environment and their condition is largely unknown. The large investment necessary to rehabilitate

the stadium warrants further assessment of the piles to gain some sense of the likelihood that pile foundation remediation will be required before the end of the useful life of the balance of the rehabilitated structure.

ILLUSTRATIONS

Photo 1

Overall view of the Miami Marine Stadium.

Photo 2

Piles showing moderate deterioration.

Photo 3

Piles showing severe deterioration with section loss.

Photo 4

Pile with severe deterioration below the water level.

Photo 5

Column at open area with large spalls.

Photo 6

Beam at open area with large spalls.

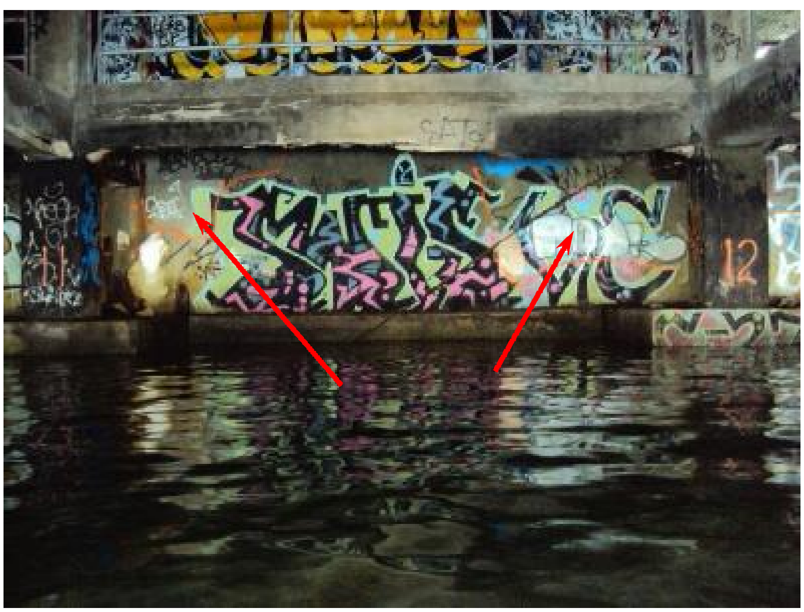

Photo 7

Seawall section with delaminated areas. Extent of delamination is bounded by the orange paint.

Photo 8

Delamination in seawall.

Photo 9

Internal crack in seawall caused by corrosion of reinforcement.

Photo 10

Deterioration on topside ground level slab.

Photo 11

Grade beam in good condition.

Photo 12

Deterioration at hanger location.

Photo 13

Deterioration on topside of ramp slab.

Photo 14

Deterioration on topside of mezzanine slab.

Photo 15

Beam and column along column line E showing severe deterioration.

Photo 16

Slab showing severe deterioration.

Photo 17

Deterioration on the topside of seating slab.

Photo 18

Deterioration on the bottom side of seating slab.

Photo 19

Crack on top side of seating slab.

Photo 20

Grout plugs missing on risers.

Photo 21

Spall on vomitory wall.

Photo 22

Deterioration on upper portion of parapets.

Photo 23

Severe deterioration on parapets around front slanted columns.

Photo 24

Moderate deterioration on raker beam.

Photo 25

Severe deterioration on beams next to the uppermost seating row.

Photo 26

Deterioration on back slanted columns.

Photo 27

Deterioration on front slanted columns.

Photo 28

Typical spall on top side of roof hypars.

Photo 29

Deterioration on bottom side of roof.

Photo 30

Typical cracking of roof hypars (bottom side).

Photo 31

Typical cracking of roof hypars (top side).

Photo 32

Deterioration at post-tensioning anchorage zone.

1993

2009

Photo 33

Piles F-1 and G-1 not showing significant deterioration growth.

1993

2009

Photo 34

Cracking on bottom side of roof near line 4 front slanted column do not appear to have increased.

1993

2009

Photo 35

Column C-1 showing some growth of spalls.

1993

2009

Photo 36

Column C-17 showing some growth of spalls.

1993

2009

Photo 37

Column E-13 showing minor growth of spalls.

1993

2009

Photo 38

Ramp next to column line 5 showing some growth of spalls.

1993

2009

Photo 39

Beams over water showing growth of spalls and new spalls.

1993

2009

Photo 40

Cracking on bottom side of roof near line 12 front slanted column do not appear to have increased.

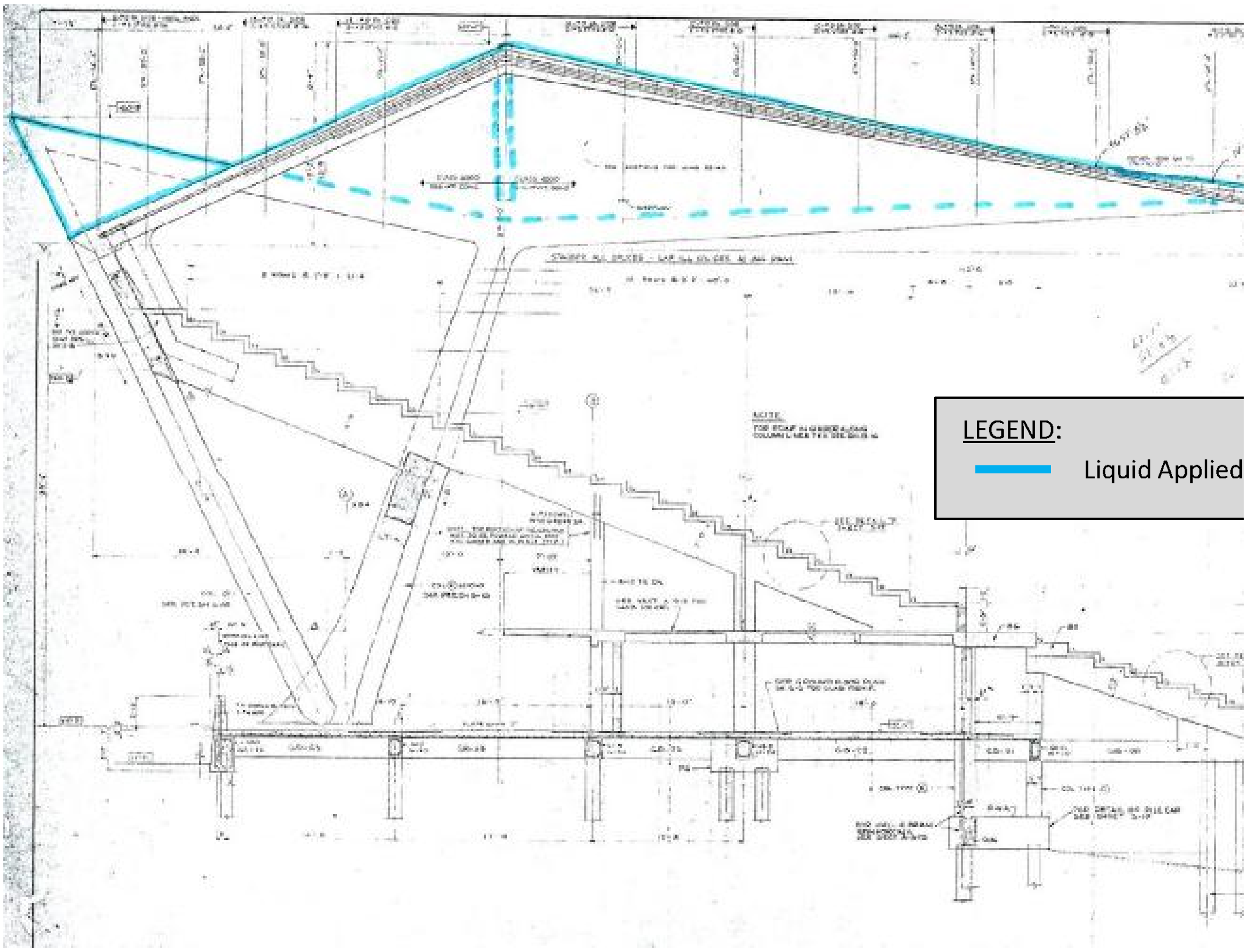

Figure 1. Concrete Protection – Alternative 1

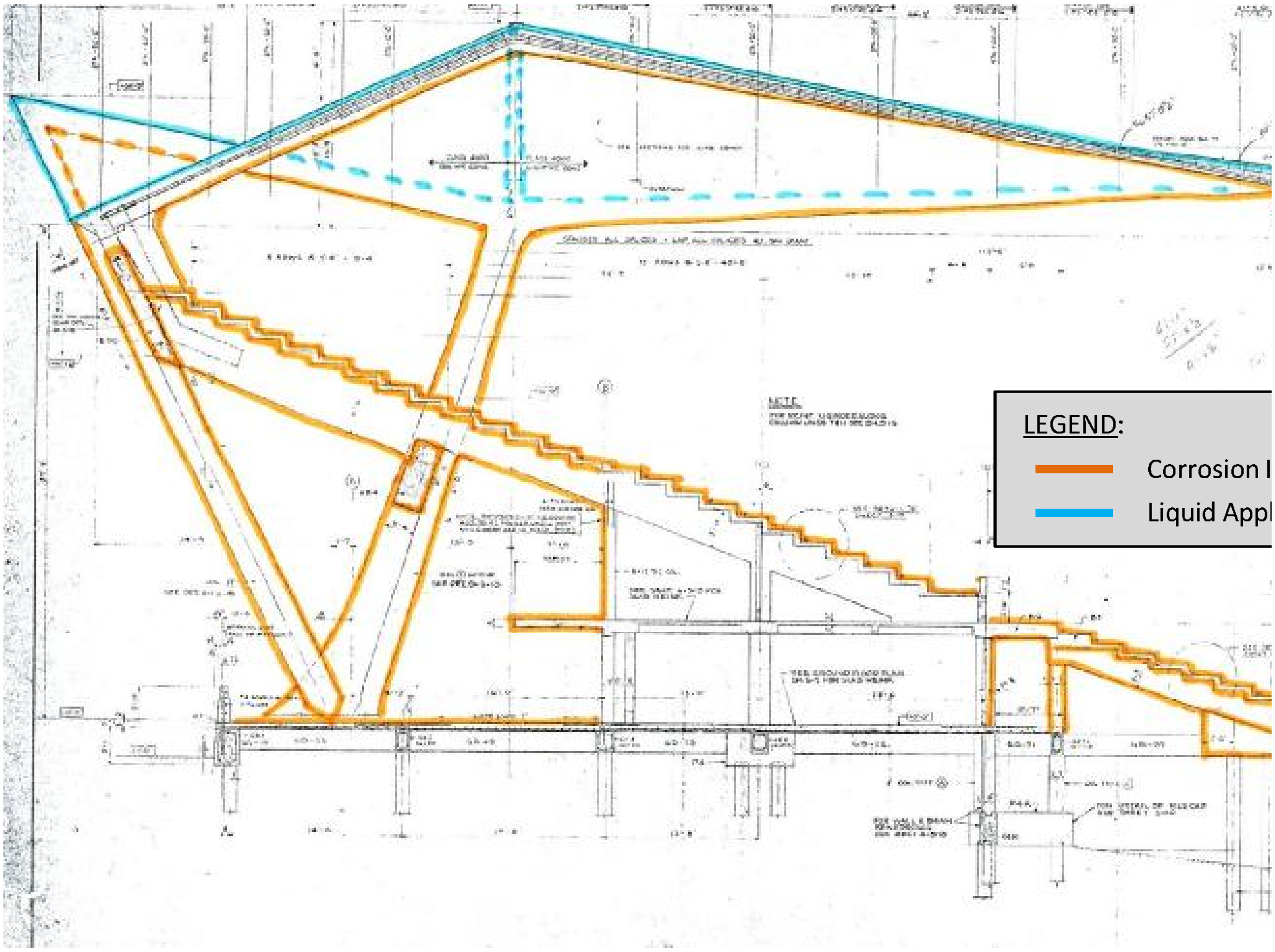

Figure 2. Concrete Protection – Alternative 2

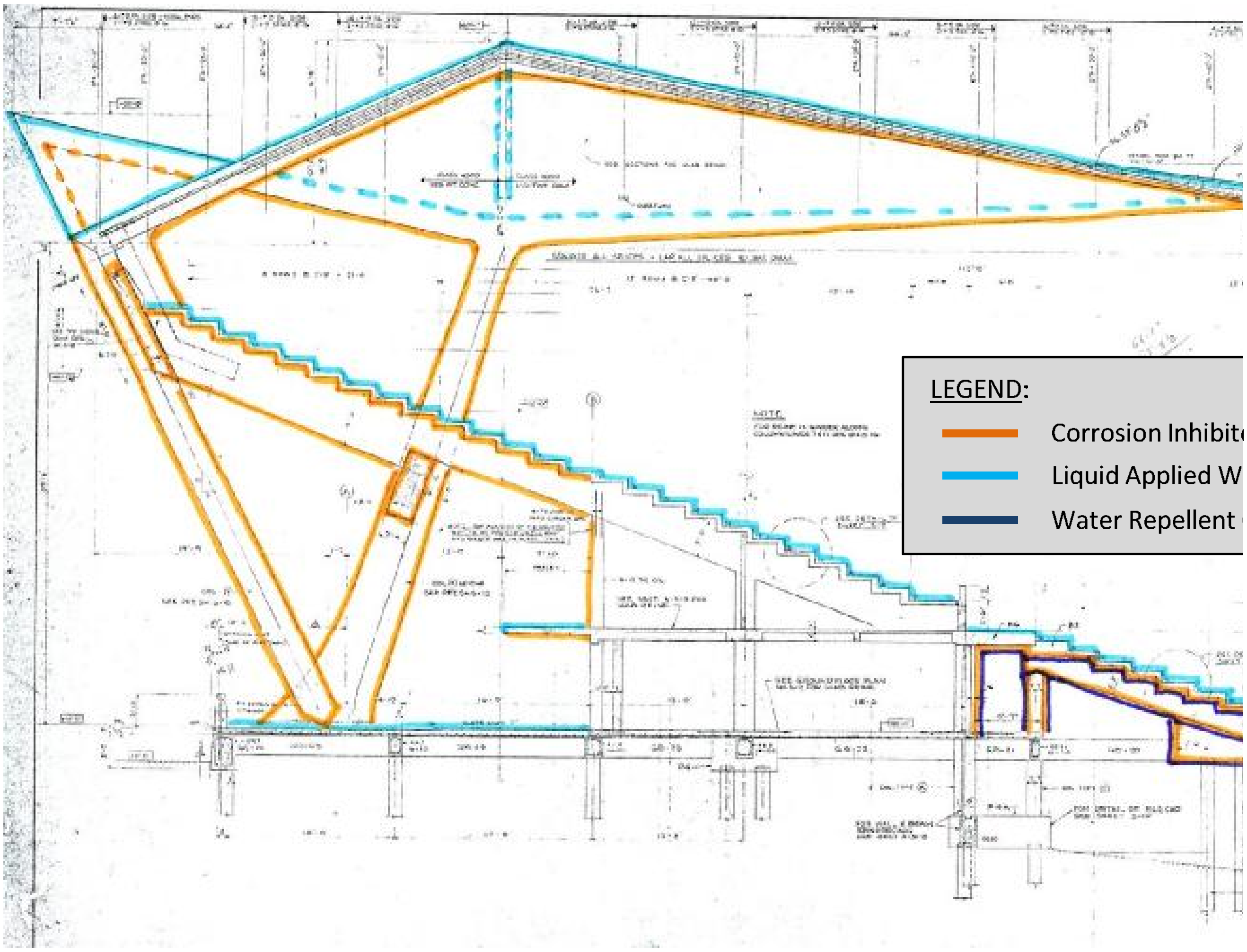

Figure 3. Concrete Protection – Alternative 3

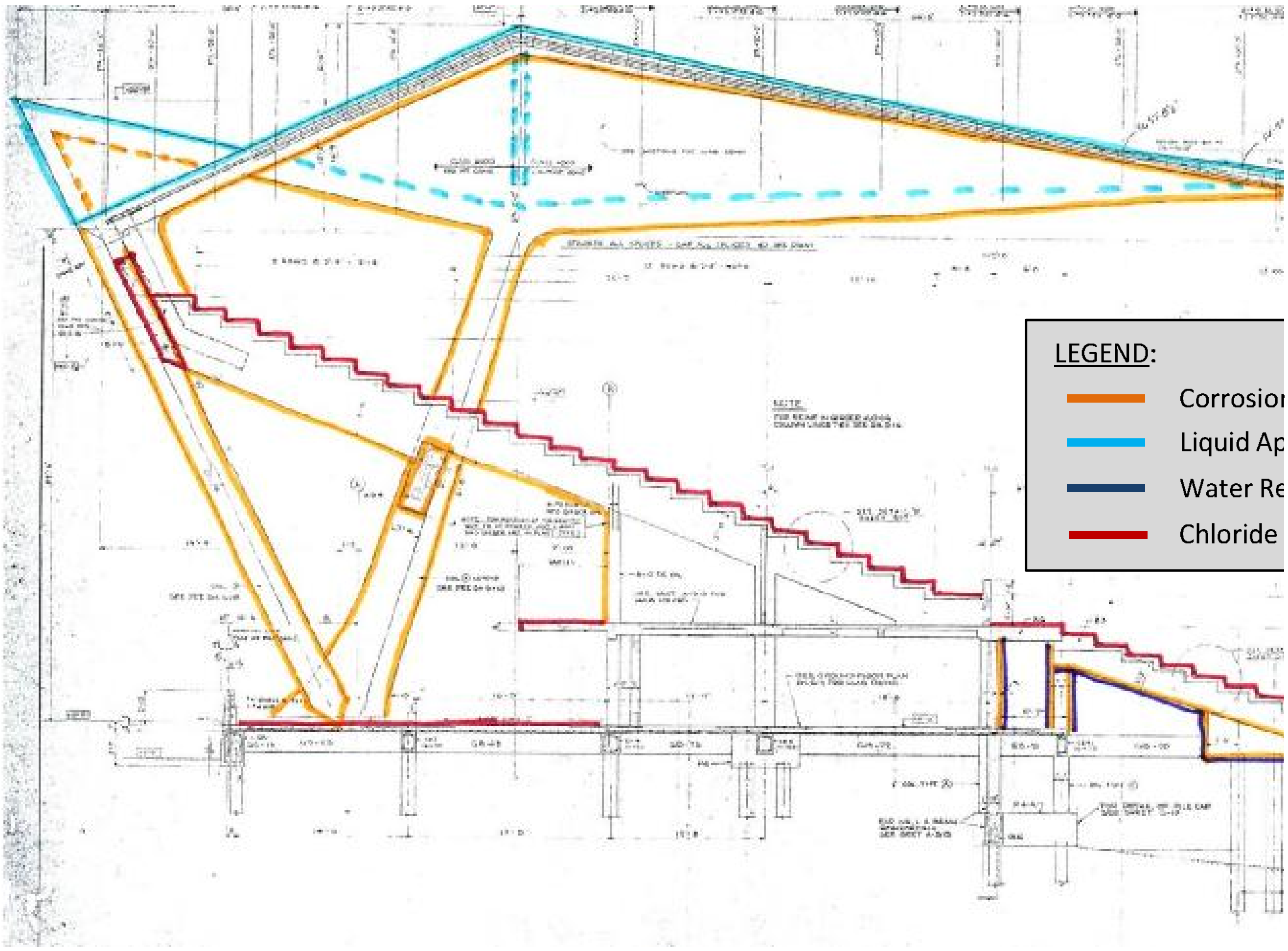

Figure 4. Concrete Protection – Alternative 4

APPENDIX

APPENDIX A

SIMPSON GUMPERTZ & HEGER

Engineering of Structures and Building Enclosures

12 October 2009

LABORATORY REPORT

BY Patrick B. Kelley

PROJECT 090457 – Estimate of the Scope and Cost of Structural Concrete Repairs, Miami Marine Stadium, Miami, FL

SUBJECT Depth of Carbonation

SAMPLES Sixteen concrete core samples were submitted by Liying Jiang on 7 October 2009.

PROCEDURES

We used a tile saw to create fresh fracture surfaces on the concrete core samples. We then applied phenolphthalein solution to the freshly cut surfaces, as noted in the attached table.

We recorded the minimum and maximum distances from the top surface down and the bottom surface up. We measured to the region in which the phenolphthalein turned magenta (pH >9), where no carbonation occurred.

RESULTS

Sample ID	Depth of Carbonation Top Down	
	Minimum (in.)	Maximum (in.)
C1	0	5/16
C3	1/16	3/8
C6	None	None
C8	0	1/2
C9	1/8	3/16
C11	None	None
C13	None	None
C14	None	None
C15	None	None
C17	None	None
C18	None	None
C19	0	9/16
C20	0	5/16
C21	None	None
C22	0	0
C23	0	1 /16

Sample ID	Depth of Carbonation Bottom Up	
	Minimum (in.)	Maximum (in.)
C1	None	None
C3	None	None
C6	0	1/4
C8	None	None
C9	0	1/8
C11	None	None
C13	None	None
C14	None	None
C15	None	None
C17	0	1/8
C18	None	None
C19	None	None
C20	0	11/16
C21	None	None
C22	0	1/32
C23	None	None

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15 October 2009

LABORATORY REPORT

BY Patrick B. Kelley

PROJECT 090457 – Estimate of the Scope and Cost of Structural Concrete Repairs, Miami Marine Stadium, Miami, FL

SUBJECT Tests to Determine Chloride Ion Contents in Concrete

SAMPLES Sixteen concrete cores were submitted by Liying Jiang on 7 October 2009.

PROCEDURES

The specimens were prepared and analyzed in accordance with AASHTO T-260-97 – The Standard Method of Sampling and Testing the Total Chloride Ion in Concrete and Concrete Raw Materials. The following summarizes our test procedures:

- We cut the samples at predetermined depths. We then crush each of the cut sections with a mortar and pestle to pass a No. 50 sieve and oven dry the samples at 150°F for a minimum of 24 hrs.
- Three grams of each sample is added to a tared 100 ml beaker and weighed to the nearest 0.1 mg.
- We add 10 ml of room-temperature distilled water to each beaker and swirl for 1 min.
- We add 3 ml of nitric acid to each beaker using a pipette and allow the sample to suspend and digest for 4 min.
- The beakers are filled to 50 ml with hot distilled water and stirred.
- We add 3 ml of hydrogen peroxide (30%) to each sample and then allow it to digest for an additional 3 min.
- Five drops of methyl orange indicator are added to each solution. When needed, additional nitric acid is added drop-wise until a slight red color is observed.
- The solutions in each beaker are covered with a watch glass and placed on a hot plate. Each solution is brought to a boil and maintained at temperature for 1 min.
- While hot, each sample is filtered through a set of double filter papers (No. 41 over No. 40) into a 250 ml flask. The filters are washed ten times with hot distilled water.

- The funnel and the outside of the filter papers are rinsed with hot distilled water. The final sample volume is approximately 150 ml.
- The samples are covered and allowed to cool to room temperature for 1-1/2 hrs.
- Each sample solution is weighed to the nearest 0.1 g using an Ohaus balance.
- Each sample solution is titrated (Gran Plot Method) using an Orion chloride-selective electrode and an Orion Model 702 Conductivity Meter.
- Silver nitrate (0.0141 N) is added to produce a reading of 225 ± 5 mV, and the amount of silver nitrate is recorded. Additional silver nitrate is added five times in 0.5 ml increments, and the resulting mV readings are recorded after each increment. A linear regression analysis of the data, the "Gran Plot Method," for each titration is then used to determine the end point and the corresponding percent chloride, which is calculated in accordance with AASHTO T260-97 – Section 5.4.2.1.

RESULTS

The chloride ion contents for each of the concrete core samples are based on an assumed total batch weight of 3,915 lbs / cu yd and a corresponding unit weight of 145 lbs / cu ft.

Sample No.	Depth From Surface	Chloride Ion	
	(in.)	%	Lbs / cu yd
C1	1/4	0.163	6.38
	3-1/4	0.041	1.61
	6	0.022	0.86
C3	1/4	0.129	5.05
	1-1/2	0.007	0.27
	7-3/4	0.078	3.05
C6	1/4	0.168	6.58
	8	0.008	0.31
	15-3/4	0.169	6.62
C8	1/2	0.032	1.25
	1-1/2	0.014	0.55
C9	1/4	0.102	3.99
	1-3/4	0.021	0.82
	3-1/4	0.022	0.86
C11	2	0.312	12.21
	3-1/2	0.638	24.98
	5-1/2	0.223	8.73
C13	1	0.346	13.55
	2	0.366	14.33
C14	1	0.728	28.50
	2	0.719	28.15
C15	2	0.644	25.21
	3-1/2	0.597	23.37
C17	1/4	0.153	5.99
	3-3/4	0.022	0.86

Sample No.	Depth From Surface	Chloride Ion	
	(in.)	%	Lbs / cu yd
	7	0.062	2.43
C18	1	0.877	34.33
	2	1.121	43.89
C19	1	0.132	5.17
	3	0.064	2.51
C20	1/4	0.079	3.09
	1-1/2	0.035	1.37
	3	0.040	1.57
C21	1	0.027	1.06
	2-1/2	0.013	0.51
C22	1/4	0.043	1.68
	1-1/2	0.031	1.21
	3	0.060	2.35
C23	1/2	0.032	1.25
	1-1/2	0.029	1.14

Note: According to ACI 201, the chloride corrosion threshold is 0.20% total acid soluble chloride by mass of cement, which for a six- or seven-bag mix is equivalent to 0.029% or 0.034%, respectively by mass of concrete, or between 1 and 1-1/2 lbs / cu yd, based on an assumed total batch weight of 3,915 lbs / cu yd.

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APPENDIX B

MIAMI MARINE STADIUM

STRUCTURAL PRESERVATION SYSTEMS

Florida Branch
2001 Blount Road
Pompano Beach, FL 33069
Phone 954-984-9555
Fax 954-984-9559
www.structural.net

Structural Repair Budget
12/31/2009

The following budget is based on conceptual repairs
Details provided by Simpson Gumpertz & Heger
Dated, 11/6/2009.

	BASE BID ITEMS				
	Work Item	**Quantity**	**u/m**	**Unit Price**	**Totals**
1	Slab Topside Repair	800	sf	$90.00	$72,000.00
2	Slab Underside Repair	150	sf	$120.00	$18,000.00
3	Slab Edge Repair	900	lf	$165.00	$148,500.00
4	Grade Beam Repair	15	cf	$520.00	$7,800.00
5	Beam Repair- Mezzanine	20	cf	$470.00	$9,400.00
6	Beam Repair- Lower Level	60	cf	$470.00	$28,200.00
7	Beam Repair- Upper Level	235	cf	$470.00	$110,450.00
8	Beam Underside Repair- Line J	400	cf	$470.00	$188,000.00
9	Upper Seat Beam Repair	200	cf	$470.00	$94,000.00
10	Vomitory Wall Repair	150	sf	$125.00	$18,750.00
11	Seating Topside Repair	1350	sf	$90.00	$121,500.00
12	Seating Topside Nosing Repair	400	lf	$110.00	$44,000.00
13	Seating Underside Repair- Lower Level	50	sf	$120.00	$6,000.00
14	Seating Underside Repair- Upper Level	750	sf	$120.00	$90,000.00
15	Column Repair- Lower Level	20	cf	$395.00	$7,900.00
16	Column Repair- Upper Level	5	cf	$395.00	$1,975.00
17	Parapet Wall Repair	300	sf	$125.00	$37,500.00
18	Seawall Repair	2100	sf	$135.00	$283,500.00
19	Pile Repair (includes cp jacket)	60	ea	$7,200.00	$432,000.00
20	New Pile Support	10	ea	$30,000.00	$300,000.00
21	Back Roof Column Repair	135	cf	$395.00	$53,325.00
22	Front Roof Column Repair	50	cf	$395.00	$19,750.00
23	Roof Topside Repair	200	sf	$150.00	$30,000.00
24	Roof Underside Repair	300	sf	$190.00	$57,000.00
25	Roof Crack Injection	10000	lf	$58.00	$580,000.00
26	Roof Diaphragm Wall Repair	50	sf	$170.00	$8,500.00
27	Roof Diaphragm Anchor Encasement	16	ea	$350.00	$5,600.00
28	Beam-Column Replacement- Line D-E Ground Floor	15	ea	$19,400.00	$291,000.00
29	Slab Replacement-Line D-E Ground Level	2000	sf	$70.00	$140,000.00
30	Beam Replacement Line E Ground Level	470	cf	$350.00	$164,500.00
31	Beam Replacement Ground Level	100	cf	$350.00	$35,000.00
32	Parapet Wall Replacement at Roof Cols.	300	cf	$280.00	$84,000.00
33	Slab Hanger Replacement	16	ea	$6,100.00	$97,600.00
34	Parge Coat Repair	500	sf	$17.00	$8,500.00
35	Underside Grout Plug Repair	600	ea	$50.00	$30,000.00
36	Topside Grout Plug Repair	600	ea	$35.00	$21,000.00
37	Crack Injection at other locations	1000	lf	$40.00	$40,000.00
38	Supplemental Reinforcing	3000	lbs	$3.80	$11,400.00
39	Graffiti Removal	50000	sf	$1.75	$87,500.00
40	Remove Seats and patch bolt holes	6500	ea	$25.00	$162,500.00
41	Scaffolding and Access	1	ls	$144,000.00	$144,000.00
42	Mobilization/Demobilization	1	ls	$25,000.00	$25,000.00
43	Environmental Protection Allowance	1	ls		$50,000.00
44	Shoring Allowance	1	ls		$100,000.00
45	Building Permit (City of Miami- 3.5%)	1	ls		$128,000.00
46	P & P Bonds- 2%	1	ls		$85,500.00
	Total Budget Estimate				$4,479,150.00
	Notes:				
1	Beam and column spall repairs were converted from sf to cf based on an ave. 6" depth				
2	All slab and wall repairs in square foot are assumed to be a maximum 3" depth				
3	All slab edge repairs are assumed to be a maximum 12" back from leading edge X the full depth of the slab				
4	One Galvashield annode is included for every cubic foot of concrete repaired/none included for new beams/columns or slabs				

MIAMI MARINE STADIUM

Structural Repair Budget
12/31/2009

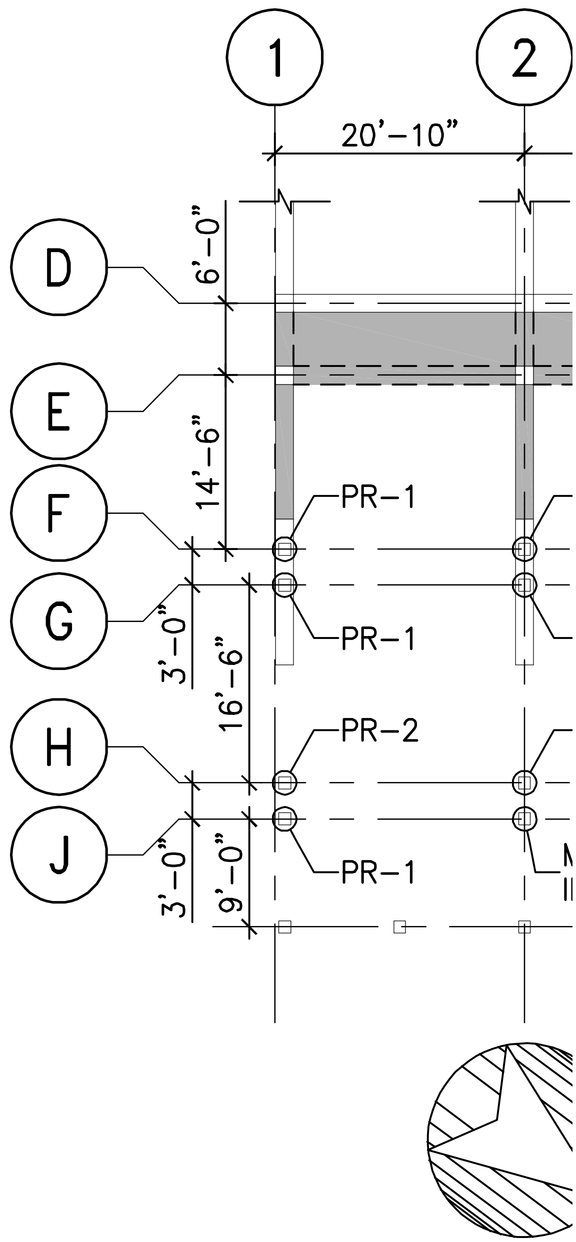
The following budget is based on conceptual repairs
Details provided by Simpson Gumpertz & Heger
Dated, 11/6/2009.

STRUCTURAL PRESERVATION SYSTEMS

Florida Branch
2001 Blount Road
Pompano Beach, FL 33069
Phone 954-984-9555
Fax 954-984-9559
www.structural.net

ALTERNATE BID ITEMS (The following prices are adds to the base bid)		
		$395,600.00
Alternate 1		
1	Liquid applied waterproofing- roof topside and diaphragm (46,000 sf)	
	Bonds and Permits	$21,758.00
	Total Alternate 1........	**$417,358.00**
		$720,000.00
Alternate 2		
1	Liquid applied waterproofing- roof topside and diaphragm (46,000 sf)	
2	Corrosion inhibitor- underside of upper seating (38,000 sf)	
3	Corrosion inhibitor- roof columns (11,500 sf)	
4	Corrosion inhibitor- roof underside (40,000 sf)	
5	Corrosion inhibitor- topside of upper seating, slabs, walls and columns (88,000 sf)	
6	Corrosion inhibitor- underside of lower seating (28,500 sf)	
	Bonds and Permits	$39,600.00
	Total Alternate 2........	**$759,600.00**
		$1,149,475.00
Alternate 3		
1	Liquid applied waterproofing- roof topside and diaphragm (46,000 sf)	
2	Liquid applied waterproofing- topside of upper seating (80,000 sf)	
3	Clear water repellent- side of columns along lines D & E (1,500 sf)	
4	Clear water repellent- underside of lower seating (28,500 sf)	
5	Corrosion inhibitor- underside of upper seating (38,000 sf)	
6	Corrosion inhibitor- roof columns (11,500 sf)	
7	Corrosion inhibitor- roof underside (40,000 sf)	
8	Corrosion inhibitor- side of columns and walls (9,000 sf)	
9	Corrosion inhibitor- underside of lower seating (28,500 sf)	
	Bonds and Permits	$63,222.00
	Total Alternate 3........	**$1,212,697.00**
		$2,735,600.00
Alternate 4		
1	Liquid applied waterproofing- roof topside and diaphragm (46,000 sf)	
2	Clear water repellent- side of columns along lines D & E (1,500 sf)	
3	Clear water repellent- underside of lower seating (7,000 sf)	
4	Corrosion inhibitor- underside of upper seating (12,500 sf)	
5	Corrosion inhibitor- roof columns (11,500 sf)	
6	Corrosion inhibitor- roof underside (40,000 sf)	
7	Corrosion inhibitor- side of columns and walls (2,500 sf)	
8	Corrosion inhibitor- underside of lower seating (5,500 sf)	
9	Chloride extraction-upper beam (7,500 sf)	
10	Chloride extraction-topside of lower and upper seating	
	Bonds and Permits	$150,458.00
	Total Alternate 4........	**$2,886,058.00**
Note:	The cost of bonds, permits and general conditions are included in each alternate, 2% and 3.5% respectively.	

DRAWINGS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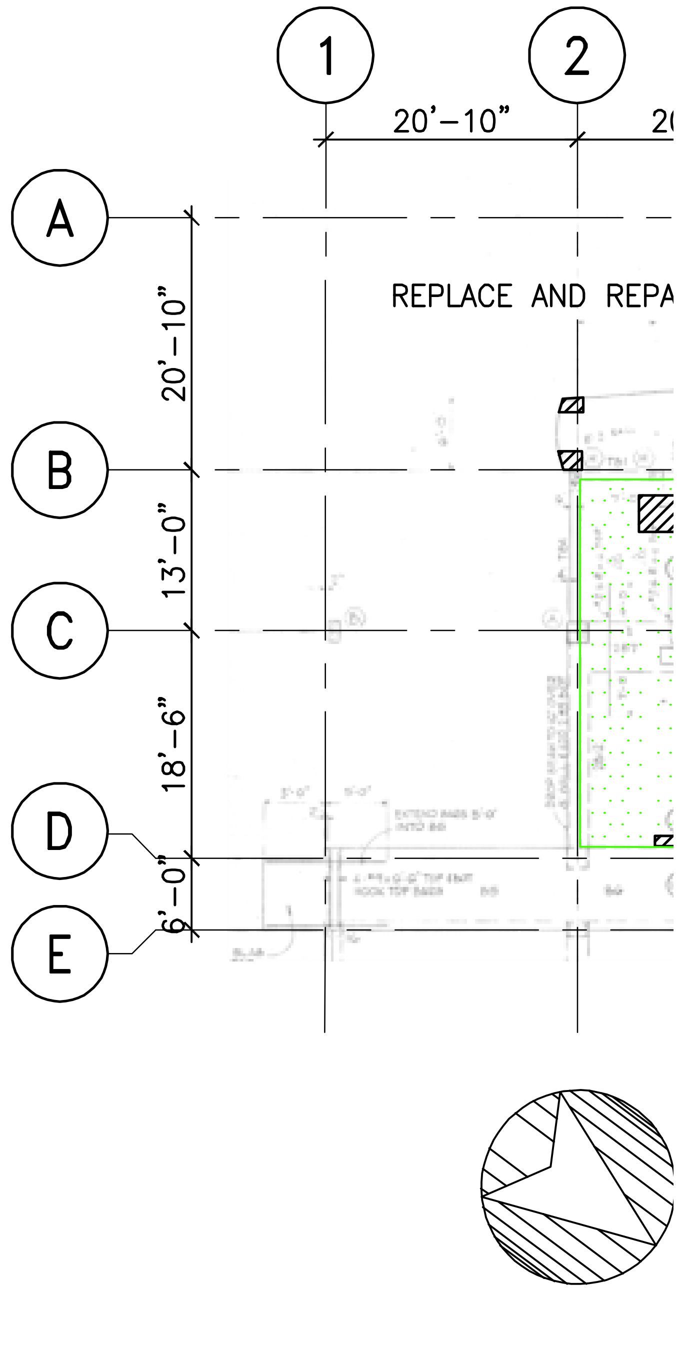
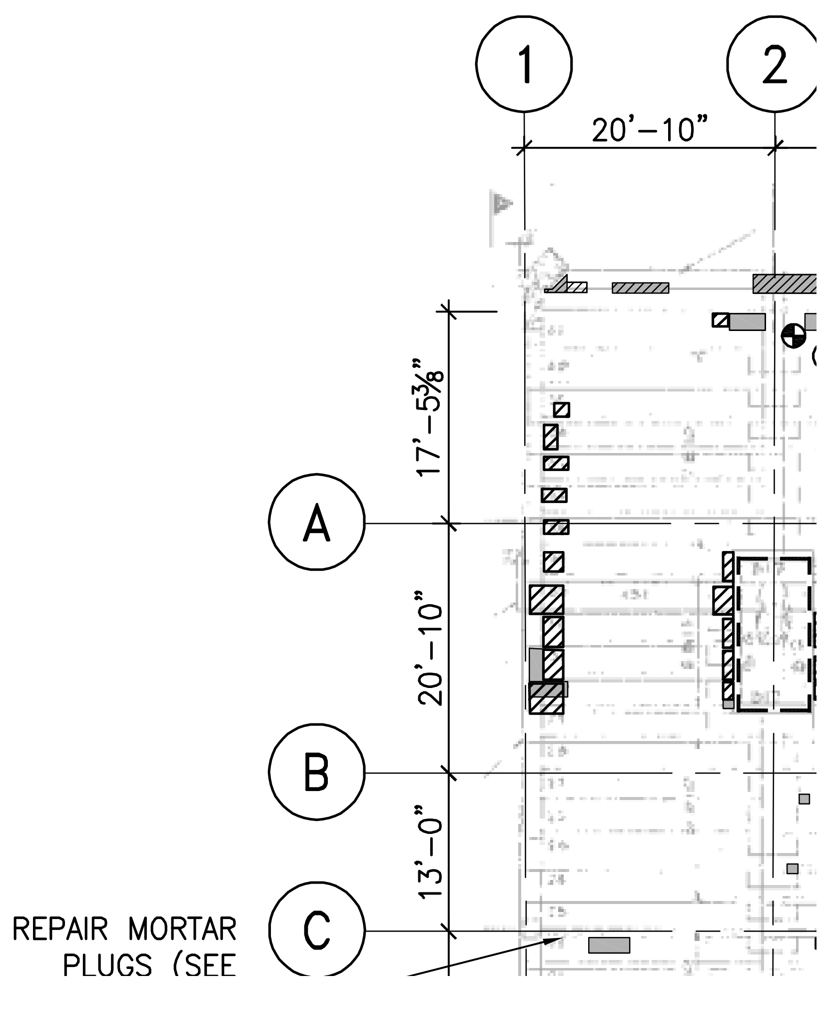
D
E
F
G
H
J
6'–0"
14'–6"
3'–0"
16'–6"
3'–0"
9'–0"
PR–1
PR–1
PR–2
PR–1



1
2
20'–10"
A
B
C
D
E
REPLACE AND REPA
20'–10"
13'–0"
18'–6"
6'–0"


1
2
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17'–5⅜"
A
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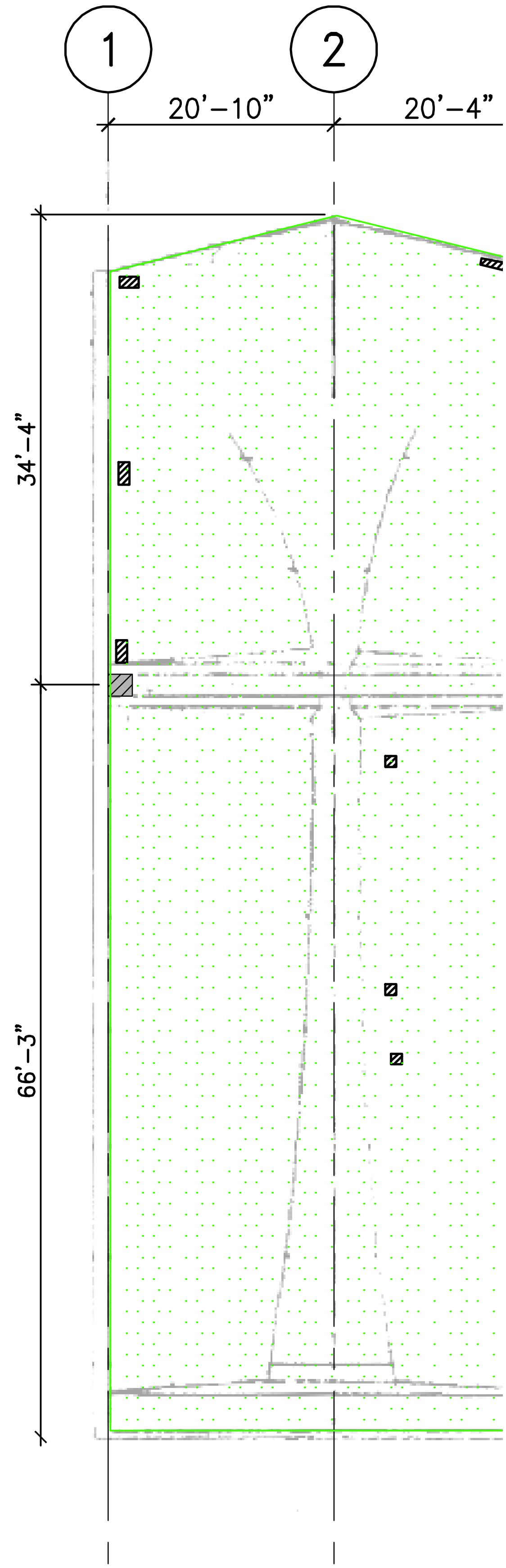
B
13'–0"
C
REPAIR MORTAR
PLUGS (SEE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2
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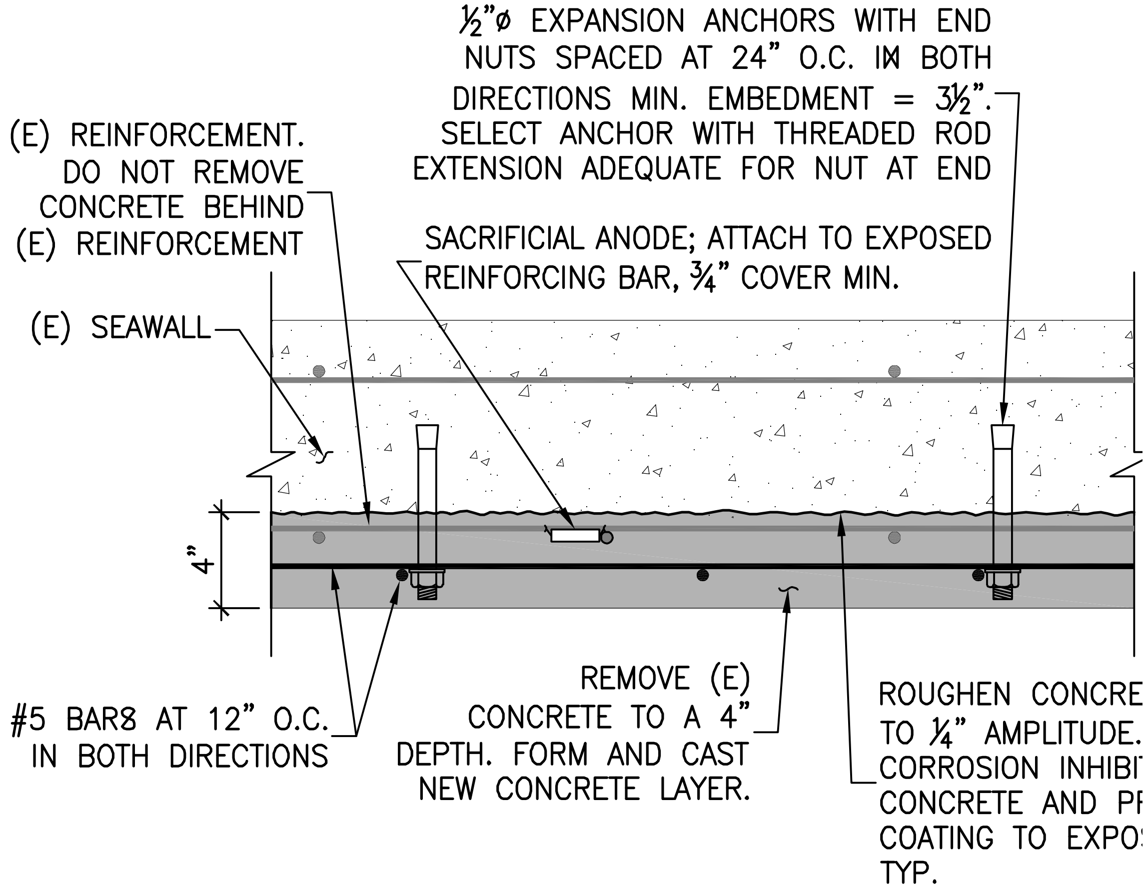
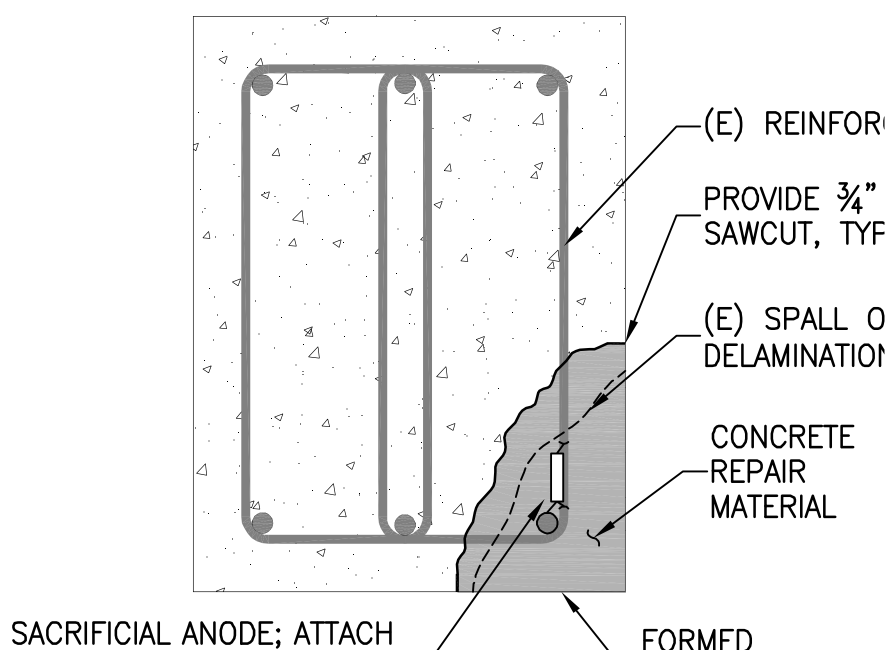
34'–4"
66'–3"

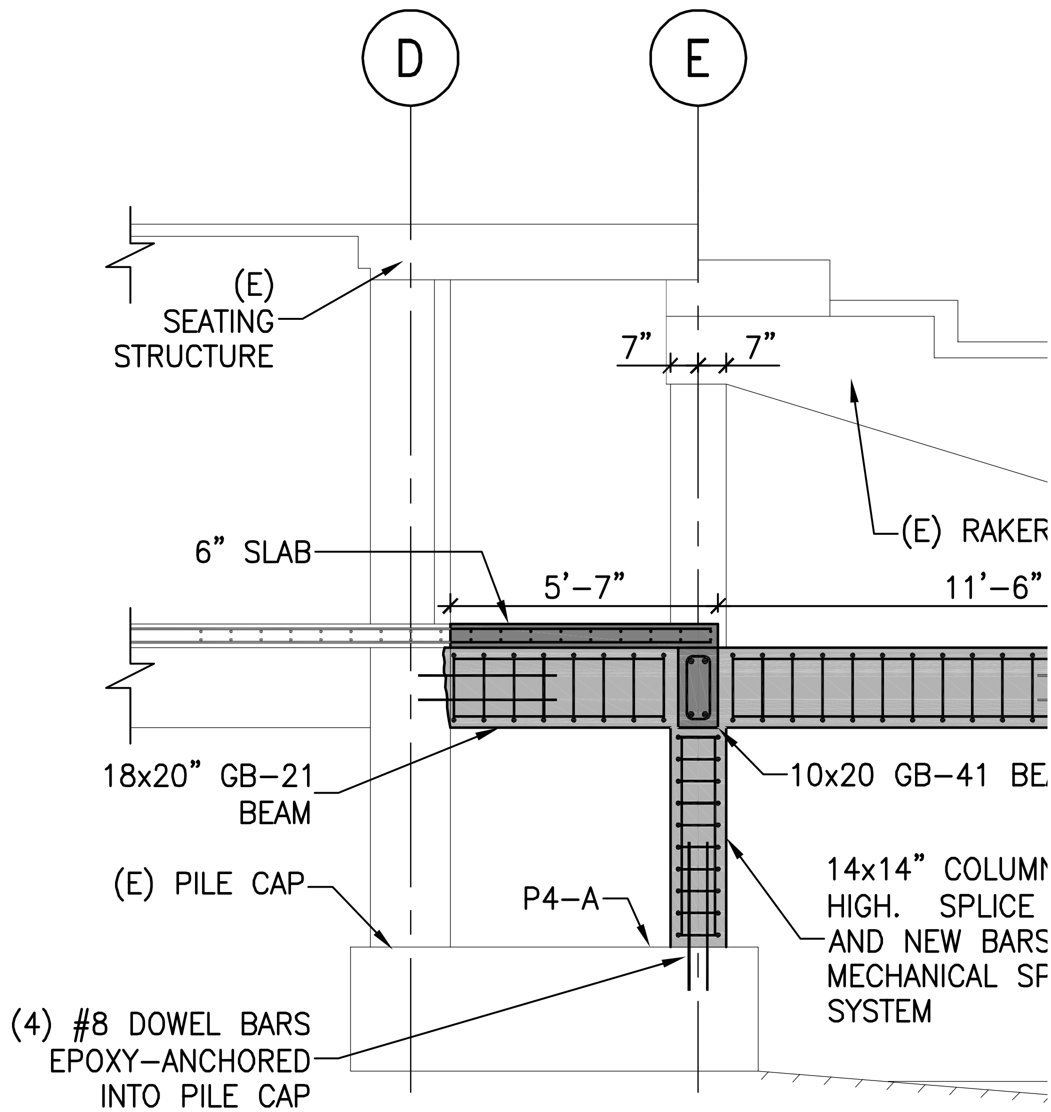


1

SEAWALL REPAIR

1–1/2





1 LINE D-E REPL