November 6, 2001

M E M O R A N D U M

TO: District Structures Design Engineers
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FROM: William N. Nickas, P.E., State Structures Design Engineer

COPIES: Secretary Tom Barry, Assistant Secretary Ken Morefield, District Secretaries (David Twiddy, Aage Schroder, Edward Prescott, Rick Chesser, Mike Snyder, Jose Abreu, Ken Hartmann, Jim Ely), Freddie Simmons, Billy Hattaway, Bill Domico, Jack Evans, Bob Nichols, Larry Sessions, Marcus Ansley, Doug Edwards (FHWA), Greg Xanders, Sharon Holmes

SUBJECT: Temporary Design Bulletins C00-5 and C01-01 (Post-Tensioning Issues) Dated September 29, 2000 and February 7, 2001, Respectively

The Florida Department of Transportation, as a result of recent findings related to corrosion of prestressing steel in post-tensioned bridges, will be establishing new policies and procedures to enhance the long-term durability of post-tensioning tendons.

In accordance with the results of the e-mail voting by FDOT District and Central Office Engineers (Technical Advisory Group - TAG) conducted through the close of business on Monday, November 5, 2001, the Structures Design Office hereby extends the requirements of the subject Temporary Design Bulletins C00-5 and C01-01 for one full year until October 31, 2002.

Additionally, I want to update each of you on FDOT's post-tensioning enhancement efforts with the following comments and information:

1. Attached to this memorandum is a copy of the new final report on the Midbay Bridge post-tensioning system inspection, tests, analyses and cost of repairs.

2. By this year's end we will have completed eighty percent of an extensive study outlining the "New Directions for Florida Post-Tensioned Bridges."

These studies provide the background documentation for the new policies and procedures to be used in Florida. The study will be presented in five volumes, with each volume focusing on a different aspect of post-tensioning. The first four volumes will be available by year's end.

In "Volume 1 - Post-Tensioning in Florida Bridges", a history of post-tensioning in Florida is presented along with the different types of post-tensioned bridges typically built in Florida. This volume also reviews the critical nature of different types of post-tensioning tendons and details a new five-part strategy for improving the durability of post-tensioned bridges.

"Volume 2 – Design and Detailing of Post-Tensioning in Florida Bridges" applies the five-part strategy presented in Volume 1 to the design of post-tensioned bridges in Florida. Items such as material specifications for enhanced post-tensioning systems, plan detail requirements for grout port and vent locations, detailing practices for watertight bridges and multi-layered anchor protection requirements are presented in this volume.

The third volume "Volume 3 - Construction Inspection of Florida Post-Tensioned Bridges" also addresses the five-part strategy for the various types of post-tensioned bridges in Florida, but from the perspective of Construction Engineering and Inspection. The various types of inspections required to fulfill the five-part strategy are presented with checklists of critical items.

"Volume 4 – Condition Inspection and Maintenance of Florida Post-Tensioned Bridges" addresses the specifics of ensuring the long-term durability of tendons in existing and newly constructed bridges. The types of inspections and testing procedures available for condition assessments are reviewed, and a protocol of remedies are presented for various symptoms found.

The fifth volume being developed is "Volume 5: Load Rating Segmental Post-Tensioned Bridges in Florida". The information in this volume will provide guidance for addressing Florida's and AASHTO's load rating requirements as they pertain to Florida's precast and cast-in-place segmental bridges.

These products will guide the Florida bridge community through the rationale for changes in the whole arena of post-tensioned bridges and in doing continued business with and for the Florida Department of Transportation. A seminar schedule will be announced around January 1, 2002 that will deal with the material and policies discussed in the first four of the above-listed volumes.

Also, for your reference and convenience, please find enclosed one copy of each of the subject bulletins.

WNN:ns

Enclosures

Florida Department of Transportation
District 3

Final Report

This report summarizes inspections, tests, analyses, and repairs of the Mid-Bay Bridge post-tensioning system.

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October 10, 2001

Mid-Bay Bridge

Post-Tensioning Evaluation

Preface

The Florida Department of Transportation did not design or oversee the construction of the Mid-Bay Bridge. The Florida Department of Transportation executed a Maintenance and Operations Contract with the Mid-Bay Bridge Authority on January 1, 1990 (modified on May 16, 1991), for the purposes of preserving this piece of infrastructure.

Disclaimer

The Draft Report was published to document progress of the inspection, testing, analysis and rehabilitation of the post-tensioning system of the Mid-Bay Bridge. Concepts, ideas, and conclusions expressed in the Draft Report were not solely those of the author. The information presented represented a summary of work performed by the others and the author. The Draft Report was a work in progress and was subject to change in all areas.

The Final Report further documents the inspection, testing, analysis and rehabilitation of the post-tensioning system of the Mid-Bay Bridge. The Final Report extends information presented in the Draft Report to include the results of additional work undertaken to rehabilitate the bridge.

Mid-Bay Bridge Post-Tensioning Evaluation

Contents

Appendix A – Sounding Results

Appendix B – Borescope Results

Appendix C – Vibration Results

Appendix D – Mag-Flux Results

Appendix E – Visual Tendon Inspection Results

Appendix F – Other Studies

F.1 Hancor Report on PE Duct
F.2 Atofina Petrochemicals Report on PE Duct
F.3 SKW/MBT Report on Grout
F.4 FDOT Report on Tensile Strength of Strand
F.5 FDOT Report on Grout
F.6 Mid-Bay Bridge Tendon Potential Test
F.7 FDOT Paper on Tendon Removal
F.8 FDOT Plans and Specs for Tendon Maintenance

Appendix G – Superstructure Analyses

Appendix H – Field Results of Repair Work

Appendix I – Substructure Analyses

Chapter 1 – Introduction

1.1 Overview

The Mid-Bay Bridge, Florida Bridge No. 570091, is a precast segmental bridge crossing Choctawhatchee Bay in Okaloosa County, Florida. The bridge carries FL 293 between US 98 near Sandestin and SR 20 east of Niceville. A location map of the bridge is given in Figure 1.1.

On August 28, 2000, during a routine inspection of the Mid-Bay Bridge, a post-tensioning tendon in Span 28 was observed to be significantly distressed. The polyethylene sheathing surrounding the tendon was cracked, exposing the tendon's high strength prestressing strands and surrounding cementitious grout. Several of the strands of the post-tensioning tendon were fractured.

Concern raised from this observation led to an immediate "walk-through" inspection to verify if other post-tensioning tendons were exhibiting similar signs of distress. A post-tensioning tendon in Span 57 was found to have failed completely at the north end of the tendon, as evidenced by pull out from the expansion joint diaphragm.

As a result of these preliminary findings, a more complete inspection, testing and analysis program was developed to identify the source and extent of corrosion in the post-tensioning tendons and to develop necessary remedial action. This report presents the findings of these inspections, tests and analyses, as well as the repairs performed.

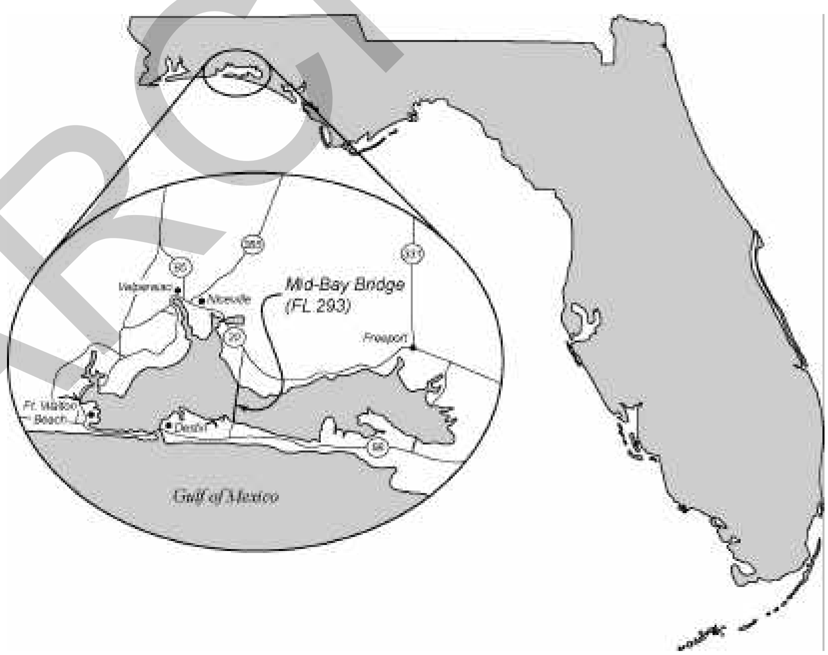


Figure 1.1 – Location of the Mid-Bay Bridge

1.2 Bridge Description

The Mid-Bay Bridge is a 19,265' precast segmental bridge crossing Choctawhatchee Bay in Okaloosa County, Florida. The bridge is made of 141 spans, arranged into 25 continuous structural units. All spans of the Mid-Bay Bridge have a length of 136', except for the 225' main span (Span 83). The alignment of the bridge is predominately south-to-north, with the beginning of the bridge (Span 1) at the southern end. The arrangement of the spans and continuous units is as follows:

Unit 1:	4 Span Unit	Spans 1 through 4
Unit 2:	5 Span Unit	Spans 5 through 9
Units 3 – 14:	12-6 Span Units	Spans 10 through 81
Unit 15:	3 Span Unit	Spans 82, 83, 84 (136', 225', 136')
Units 16 – 23:	8-6 Span Units	Spans 85 through 132
Unit 24:	5 Span Unit	Spans 133 through 137
Unit 25:	4 Span Unit	Spans 138 through 141

The typical cross section of the precast segmental superstructure is the single cell box girder shown in Figure 1.2. The bridge has an out-to-out width of 42'-9" and has a depth of 8'. The roadway width between the barrier rails is 40', providing a 12' vehicular lane and 8' shoulder in each direction of travel.

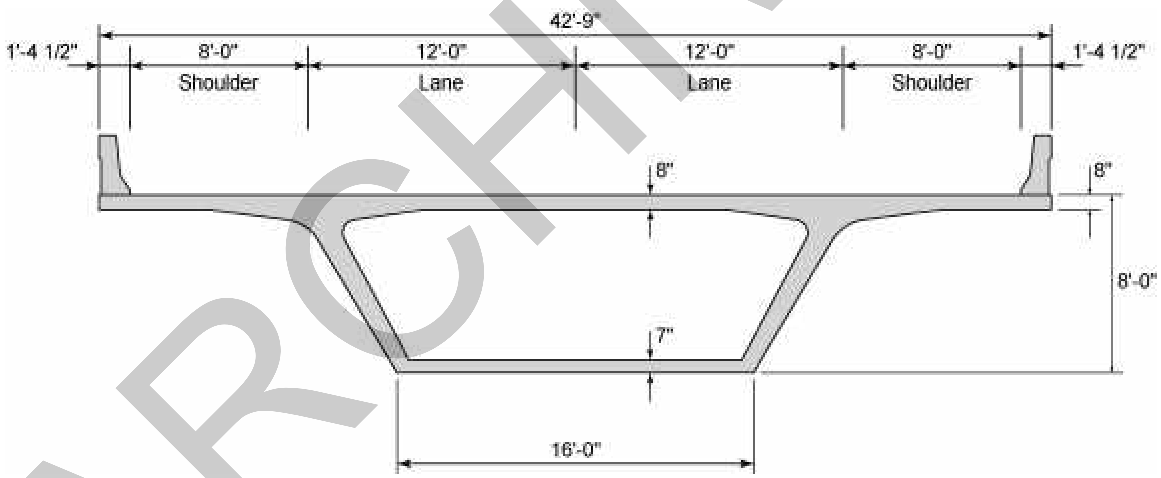
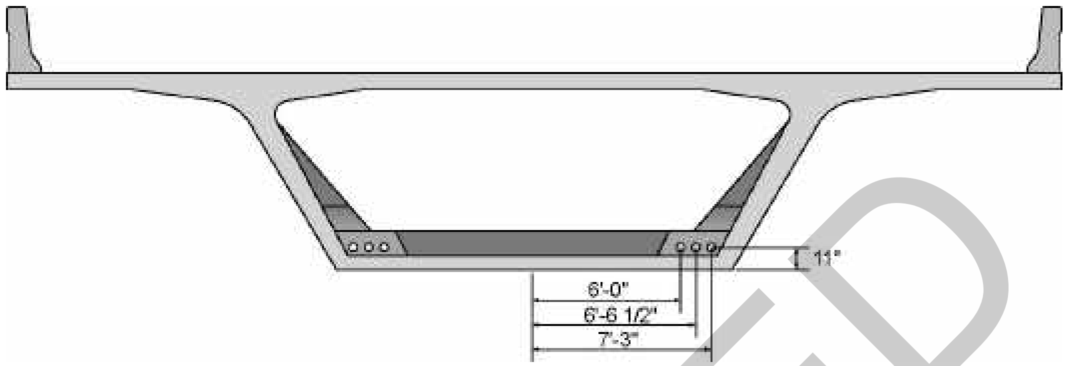
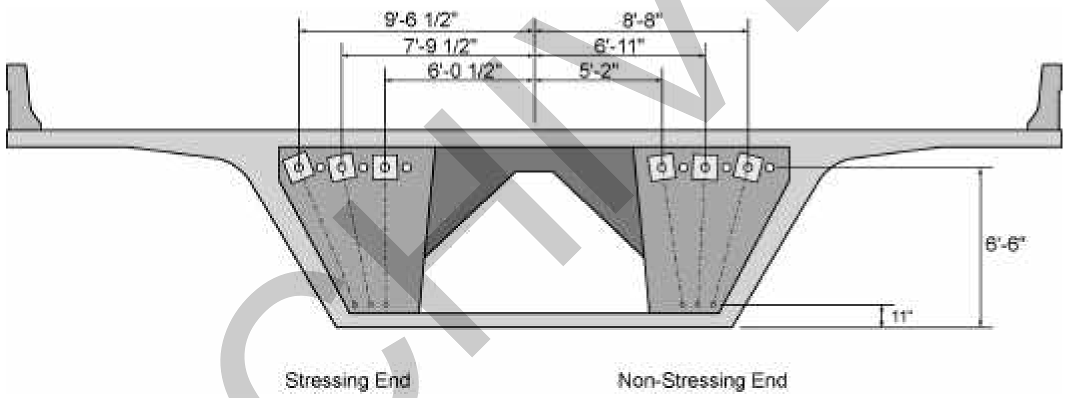


Figure 1.2 – Typical Superstructure Cross Section

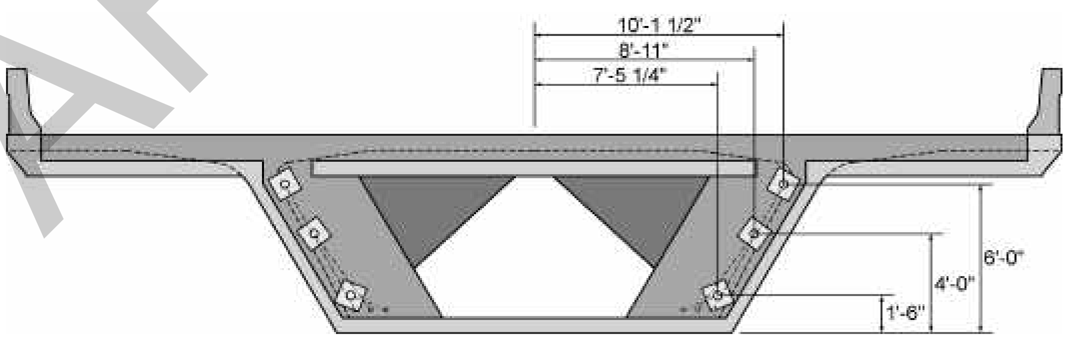
The 136' spans of the Mid-Bay Bridge are made of four types of precast segments: Typical Segments, Deviation Segments, Pier Segments and Expansion Joint Pier Segments. Each span has five, 17'-9" long typical segments with a cross section as shown in Figure 1.2. Two deviations segments in each span have the same cross section as the Typical Segments and are also 17'-9" in length. The Deviation Segments contain concrete diaphragms and bottom slab beams to transfer the force of the longitudinal post-tensioning as it changes profile within the span. Pier Segments are 10'-9" in length, are placed symmetrically over the interior piers, and contain diaphragms to anchor post-tensioning tendons and transfer superstructure forces to the substructure. Two, 5'-3", Expansion Joint Pier Segments are required at each expansion joint pier between continuous units. The Expansion Joint Pier Segments also anchor post-tensioning tendons and transfer superstructure forces through internal diaphragms. The cross sections of the Deviation Segments, Pier Segments, and Expansion Joint Pier Segments are shown in Figure 1.3. Typical and Expansion Joint Span layouts are shown in Figure 1.4.



Deviation Segment



Pier Segment



Expansion Joint Pier Segment

Figure 1.3 – Segment Types and Post-Tensioning Dimensions

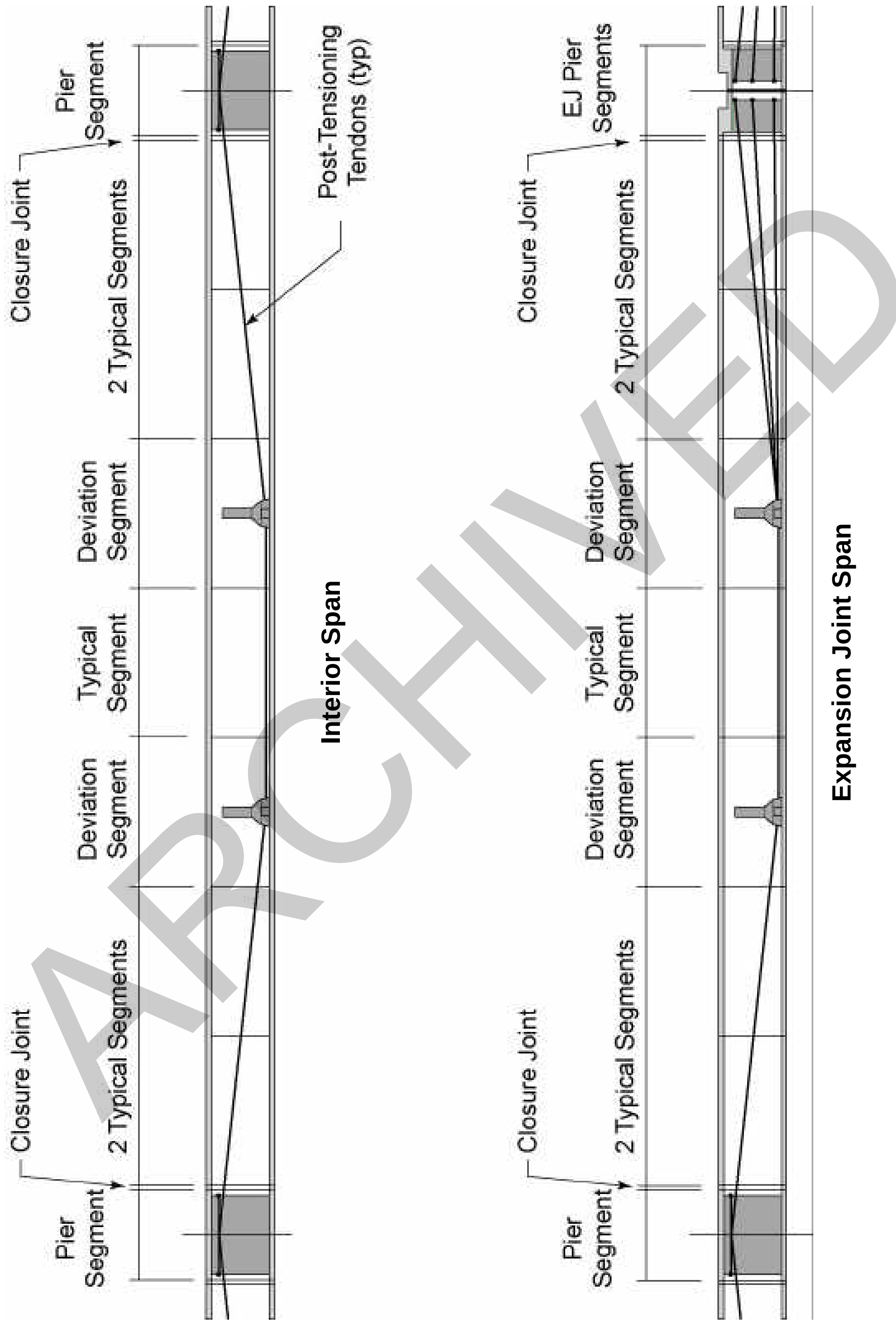


Figure 1.4 – Interior and Expansion Joint Span Layouts

Six post-tensioning tendons support each of the 138 approach spans of the Mid-Bay Bridge. Each tendon is made of 19, 0.6” diameter, 7-wire prestressing strands with a guaranteed ultimate strength of 270 ksi. The post-tensioning tendons are full span in length, anchored in either a pier or expansion joint diaphragm at the ends of the spans. The tendons deviate vertically at the deviation diaphragms to produce a “draped” profile.

The post-tensioning tendons are encased in steel ducts as they pass through the pier segment diaphragms and the deviation diaphragms. The pier segments provide a 9’-5” long embedment of the post-tensioning tendon in the pier segment diaphragms. The profile of the tendons in the pier segments begins horizontal and then curves downward to give the inclination of the draped tendon geometry. The tendon “high point” in this configuration is distributed over the approximately 4’-6” of tangent duct. The expansion joint diaphragms provide a 3’-10” embedment. The tendon is inclined all of the way through the diaphragm to the anchor. The high point of the expansion joint tendons is at the anchor.

When the tendons are external to the concrete, outside of the concrete but inside the box girder, they are placed in polyethylene ducts for protection. After the post-tensioning tendons are placed and stressed, the ducts are injected with cementitious grout for corrosion protection and bond development in concrete diaphragms.

The typical spans of the Mid-Bay Bridge were built using the Span-by-Span method of construction. In this method, an erection truss temporarily supports all segments of a span while post-tensioning is installed and stressed. A typical erection cycle would begin with the advancement of the erection truss from the previously completed span. The pier segment at the beginning of the span was assembled in the previous phase. The pier segment at the end of the span was supported on the erection truss and the next pier. Once the truss was in place, the typical and deviation segments were positioned on the trusses and aligned to the appropriate geometry. Next the post-tensioning tendons were installed and blocking placed in the closure joints between typical segments and pier segments. A small amount of post-tensioning was stressed to prevent the relative movement of the segments so that the concrete closure joints could be cast between pier segments and the first typical segments. When the closure joint concrete reached appropriate strength, the remainder of the post-tensioning was stressed and the tendons grouted, completing the erection cycle.

The direction of erection of the span-by-span construction of the Mid-Bay Bridge was from south to north for Spans 1 to 81, and from north to south for Spans 141 to 85 (descending order). Spans 82, 83, and 84 make up a continuous three span main unit containing the main span. This main unit was built using modified balanced cantilever segment construction.

Segments of precast segmental bridges can be joined with epoxy to aid in the alignment of the segments and help improve the water-tightness of the box girder. The spans of the Mid-Bay Bridge that were built using the span-by-span method of construction did not use epoxied joints. The segments of the three-span main unit, built in modified balanced cantilever, were joined with epoxy.

1.3 Project Timeline

The inspection, testing, analysis and rehabilitation activities presented in this report primarily occurred between August 28, 2000 and July 20, 2001. FDOT inspectors were performing an annual inspection of the bridge on August 28, 2000 when broken wires in Tendon 6 of Span 28 and the failure of Tendon 1 in Span 57 were discovered. The bridge was previously inspected in May of the same year, with no significant findings of distress.

Figure 1.5 shows a bar chart schedule of the inspection and initial repair activities at the Mid-Bay Bridge. These inspection and remedial actions took place immediately following the discovery of the failed post-tensioning. Crews worked multiple shifts around the clock in order to assess the condition of the bridge. In addition to other items requiring maintenance, the initial inspections revealed that 11 of the span-by-span tendons needed to be replaced.

Traffic interruptions or limitations were imposed as a result of the findings of the inspection and initial repairs performed. The following list gives the dates of traffic limitations and the impact to traffic:

August 28, 2000 and August 29, 2000	Complete Closure
August 29, 2000 to September 27, 2000	2 Axle Vehicles Only
September 27, 2000 to October 11, 2000	Complete Closure
October 11, 2000 to November 16, 2000	2 Axle Vehicles Only
November 16, 2000 to Present	All Legal Loads, no Permitted Loads

Other traffic impacts that were associated with specific tendon removal include: *

Span 28 Tendon 6	8 hour nighttime closure
Span 58 Tendon 5	1 hour daytime closure
Span 69 Tendon 3	1 hour daytime closure
Span 63 Tendon 6	1 hour daytime closure
Span 69 Tendon 2	1 hour daytime closure
Span 64 Tendon 1	1 hour daytime closure
Span 58 Tendon 6	1 hour daytime closure
Span 48 Tendon 5	1 hour daytime closure

*Tendons 57-1, 57-2 and 9-1 were replaced during bridge closures and did not require additional maintenance of traffic operations. See Section 1.4 below for tendon numbering conventions.

Subsequent to the initial actions taken to inspect and repair the Mid-Bay Bridge, presented in Figure 1.5, additional remedial actions were taken, as detailed in Chapter 4 of this report. In addition, contract plans were developed and a contract let to wrap all external tendon ducts not previously wrapped. Figure 1.6 shows an overall timeline in bar chart format for all work performed on the Mid-Bay Bridge. Figure 1.5 is a subset of Figure 1.6.

1.4 Tendon Numbering Convention

A standardized numbering scheme for the post-tensioning tendons of the Mid-Bay Bridge was adopted to organize inspection and modification efforts. This numbering scheme is shown for typical piers in Figure 1.7 and for expansion joint piers in Figure 1.8.

Example: Tendon 1 of Span 57 is referred to as Tendon 57-1.

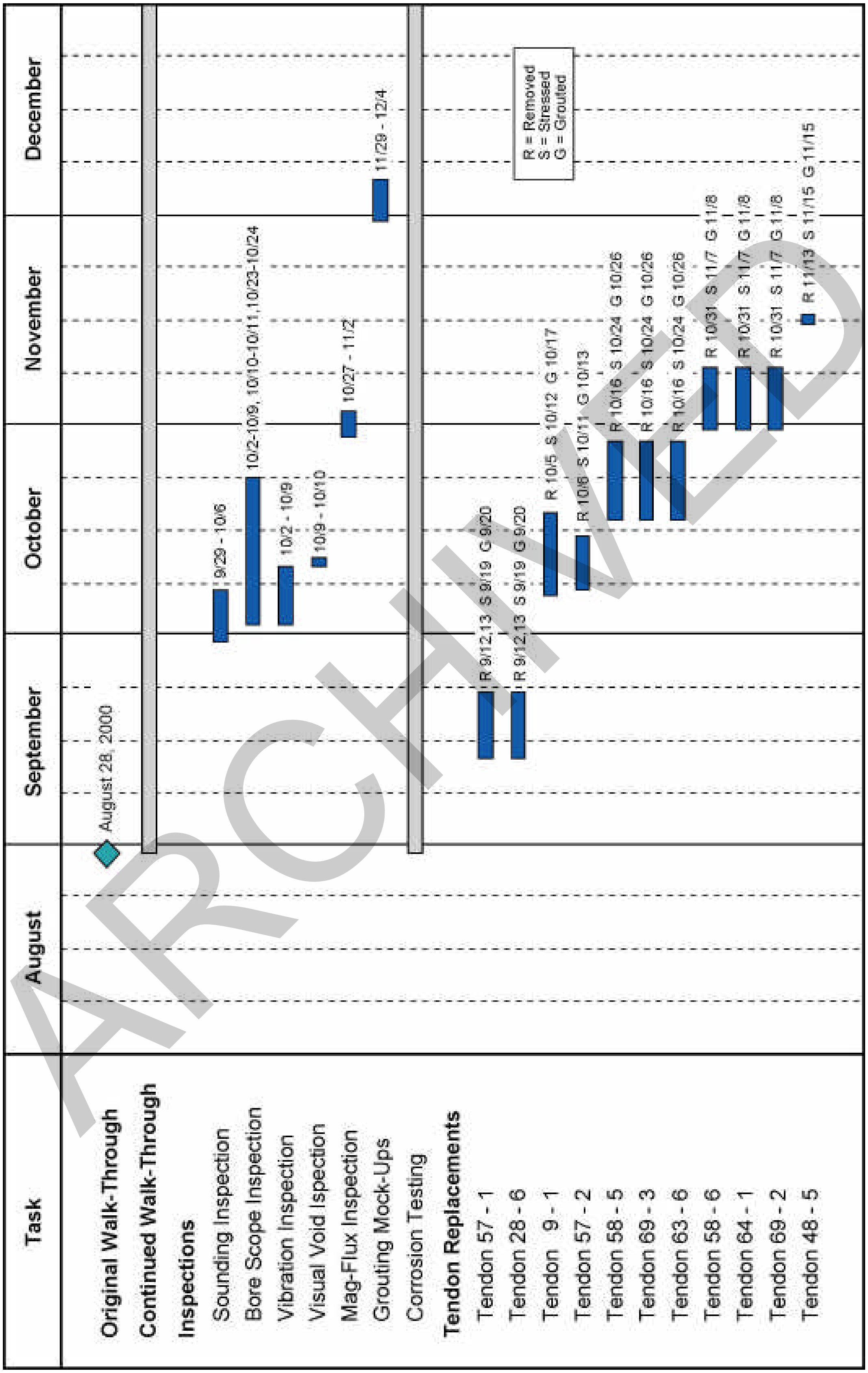


Figure 1.5 – Schedule of Inspection and Tendon Replacement for the Mid-Bay Bridge

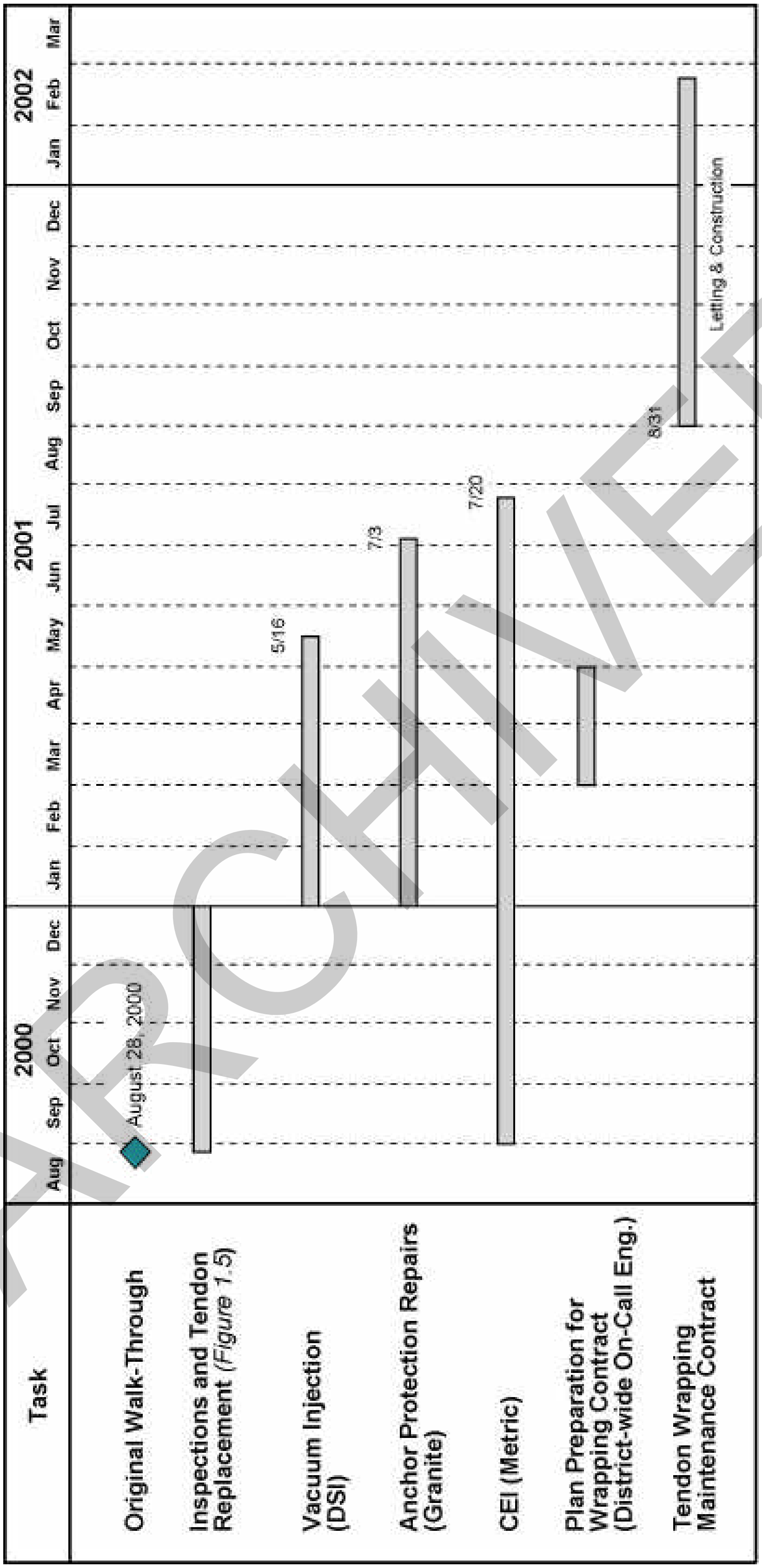


Figure 1.6 – Overall Schedule of Inspection and Repair Activities for the Mid-Bay Bridge

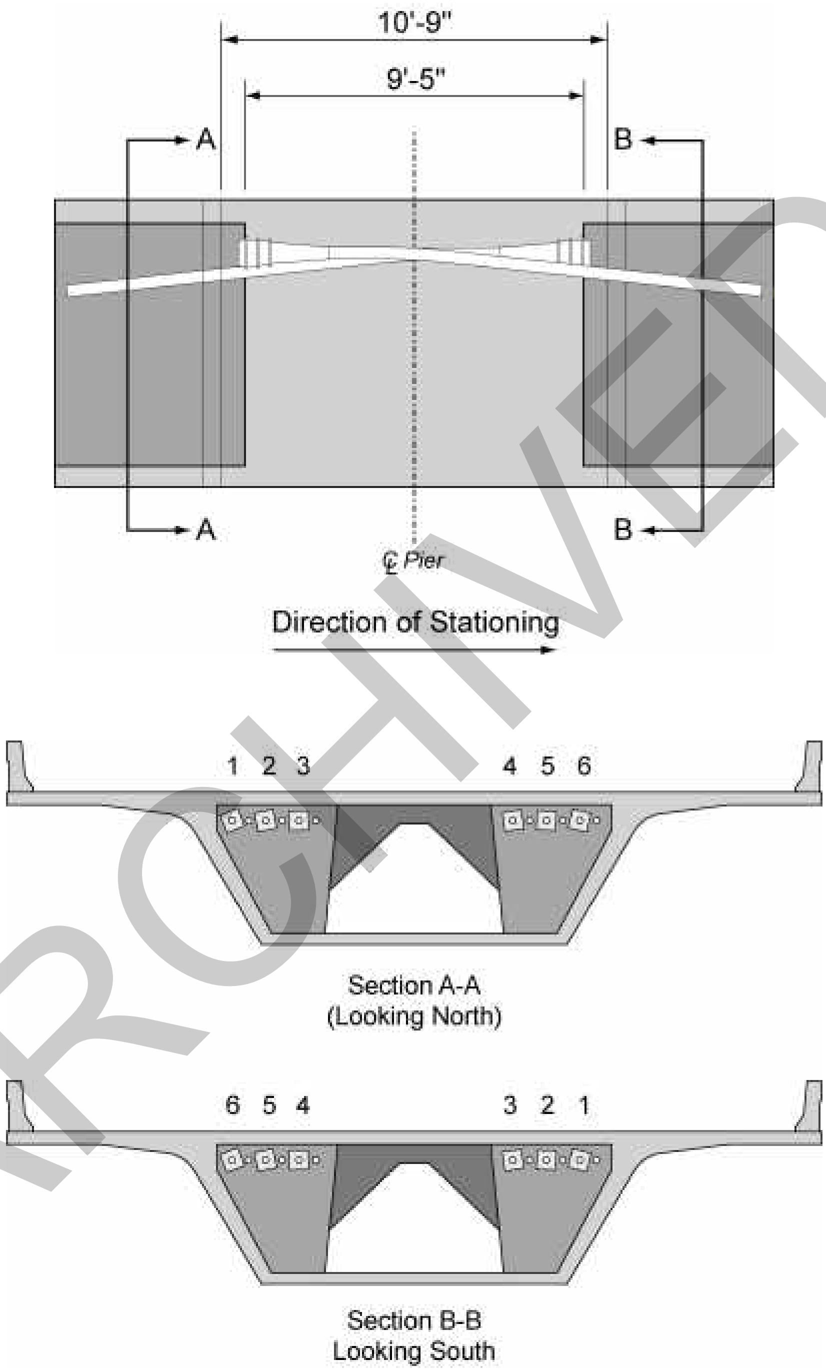


Figure 1.7 – Tendon Numbering Convention (Interior Piers)

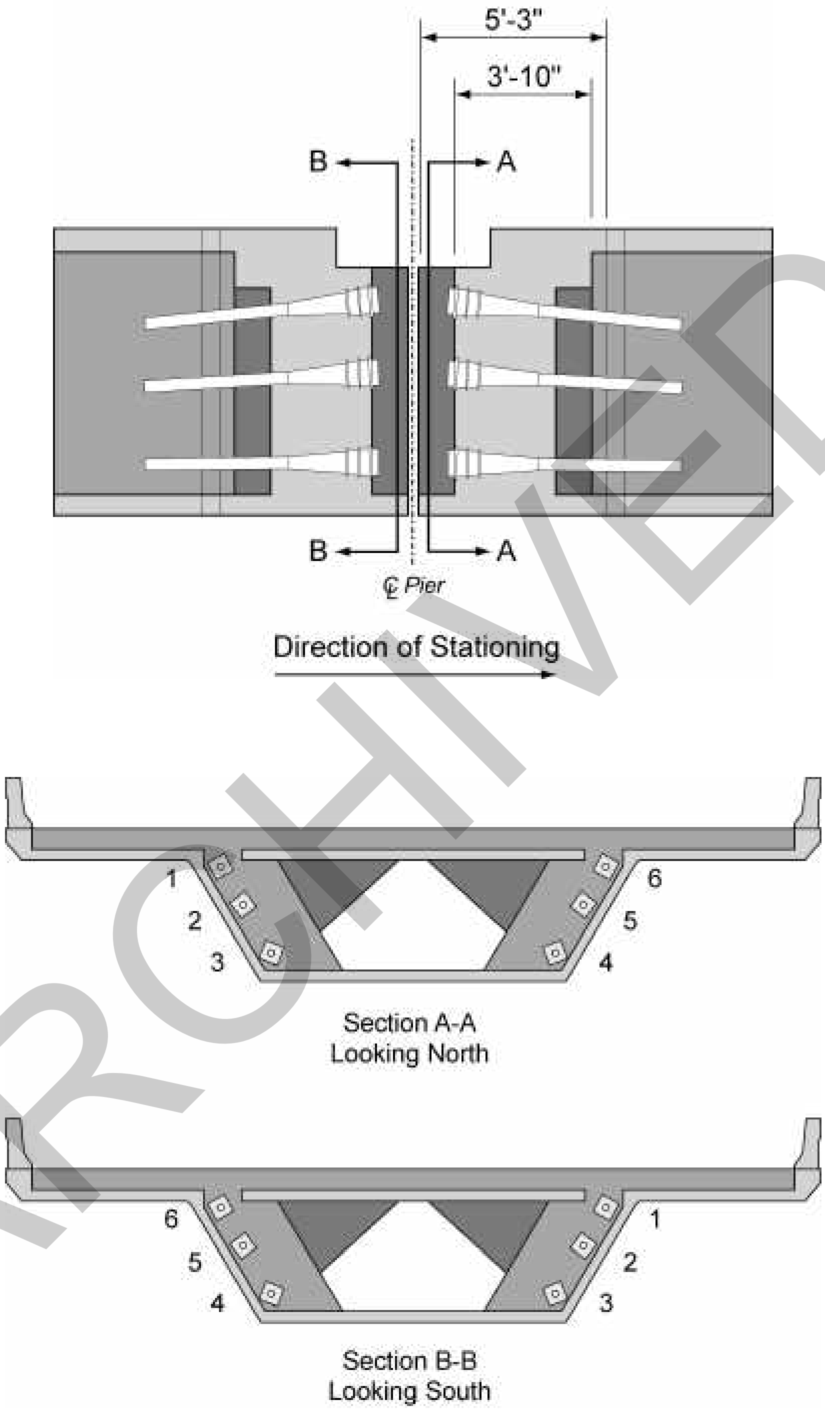


Figure 1.8 – Tendon Numbering Convention (Expansion Joint Piers)

Chapter 2 – Inspection And Testing

2.1 Introduction

The post-tensioning system of the Mid-Bay Bridge has been subjected to a rigorous inspection and testing regiment since the discovery of failed external post-tensioning tendons. The Florida Department of Transportation (FDOT) and consultant inspection personnel have worked systematically and aggressively to catalog the condition of the bridge's post-tensioning system. The inspections and tests conducted were:

- Sounding Post-Tensioning Tendons for Voids
- Borescope Inspections of Post-Tensioning Anchors
- Vibration Testing
- Visual Void Inspections
- Mag-Flux Testing
- Grouting Mock-Up Tests
- Other Corrosion Related Testing

No single inspection or testing procedure is able to provide a complete evaluation of the corrosion of external post-tensioning tendons. Some tests that produce good results in the free length of external tendons do not produce any results in the anchorage zones. Tests that produce strong indications of active corrosion in a length of tendon do not necessarily predict the level of force in the tendon or section loss that has occurred. The proper approach for inspecting external post-tensioning tendons, therefore, is to conduct a battery of tests specifically chosen to develop an understanding of the tendon conditions. This was effectively accomplished for the Mid-Bay Bridge.

This chapter summarizes the testing and inspection of the post-tensioning system of the Mid-Bay Bridge. Complete inspection reports and field data of these inspections and tests are provided in the Appendices of this report.

2.2 Sounding Post-Tensioning Tendons For Voids

Examination of the two failed tendons, one in Span 28 and one in Span 57, revealed that the condition of the grout for these tendons was of suspect quality. Air cavities, bleed water trails and soft, chalky grout characteristics were observed. Significant voids in grout or a highly porous grout can reduce the corrosion protective capabilities of the post-tensioning system. For this reason, a program of sounding the tendons for voids was initiated. In this procedure, an inspector walked the length of each tendon tapping lightly with a small tack hammer, listening for changes in the resulting sound. Locations of variations in sound that would imply a void were recorded. Locations where significant variations in sound are found were then evaluated to determine if a subsequent visual inspection was required.

Field notes of the sounding inspections for the Mid-Bay Bridge are found in Appendix A. These results show a consistent presence of voids in the tendons, though no significant distress was found during this inspection. The voids appear to be the result of air entrapped during grouting, expansive gasses produced within the grout, and formation of bleed water trails prior to grout set. Figure 2.1 shows a large void found in Tendon 37-4.

Inspectors generally agreed that sounding inspections were ineffective for determining tendon defects. Slight delaminations between the polyethylene pipe and grout, possibly caused by grout subsidence or shrinkage, often gave the indication of voids even though the duct may have been well grouted. Subsequent opening of the polyethylene duct exposed prestressing strands to the surrounding humid atmosphere. Finally, ducts opened for visual inspection required wrapping immediately after the inspection.

Figure 2.1 – Bleed water trail in Tendon 37-4 found by sounding inspection.

2.3 Borescope Inspections

The failure of Tendon 57-1 (Section 3.1) resulted from the corrosion of the prestressing strands inside the post-tensioning anchorage assembly. It was observed during removal of this tendon that there was no grout inside the anchor. At the time of inspection, the anchorage components and exposed strands were found to be dry. It was therefore concluded, that water present at the time of construction had caused the corrosion. As a result of this finding, an inspection procedure was developed using flexible borescopes to video record the conditions inside all anchors that contained voids. The inspection would determine whether sufficient grout was present in each anchor and the extent of corrosion on exposed prestressing strands.

Figure 2.2 shows the type of anchor used in the post-tensioning system in the Mid-Bay Bridge. The anchorages used on the Mid-Bay Bridge hold 19 strands of 0.6” diameter; the anchor shown in Figure 2.2 holds 12 strands. This cutaway view shows the cast metal multi-plane anchorage, the prestressing strands inside the anchorage, and the anchor plate used to hold the prestressing strands after stressing through the aid of wedges. The grout port is the larger threaded hole at the top of the multi-plane anchorage.

In the typical grouting operation of the Mid-Bay Bridge, the grout was injected through the grout port in the anchor at one end of the tendon. The grout was continuously placed until the duct between the anchors was filled and grout was flowing from the grout port at the far end of the tendon. Discharging grout in this fashion was an attempt to verify that the tendons were being completely filled with good grout. Voids found in the Mid-Bay Bridge were inside the anchor, just behind the anchor plate, extending variable distances along the length of the tendon.

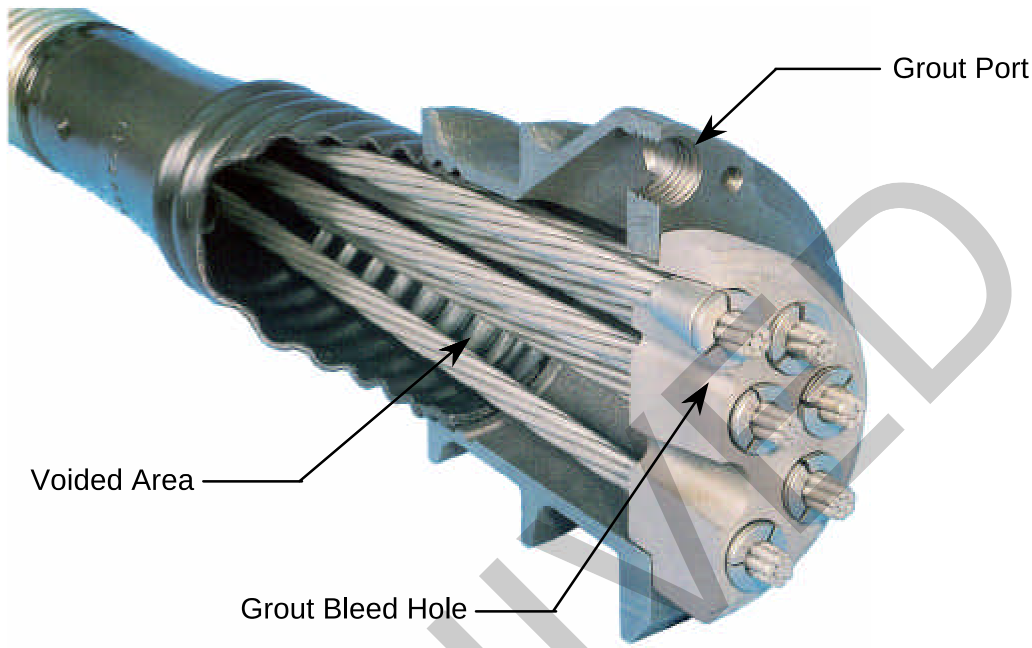


Figure 2.2 – Typical Post-tensioning tendon anchor.

Four, four-person inspection crews conducted borescope inspections of the post-tensioning anchors of the Mid-Bay Bridge. The borescope was inserted into the grout port after grout, if present in the port, was removed with a drill. Drilling was typically required for 2” to 3” before the clear access was available to a voided trumpet. If grout was found after drilling 4” then the anchorage was typically full. One inspector manipulated the borescope inside of the anchorage while another inspector controlled the video recording equipment. The other two members of each team provided support services to the two inspectors, including keeping a hand written log documenting the number of strands viewed, depth of void, extent of corrosion and whether or not a second borescope inspection was in order. For consistency in interpreting the borescope findings, the lead inspector reviewed all tendons that were recommended for a second inspection. Figure 2.3 shows a borescope team inspecting an anchorage of the Mid-Bay Bridge. The inset photograph in Figure 2.3 shows the borescope as it is inserted into the grout port.

The results of the borescope investigations are photographs and videotape recordings taken within each anchor plus field notes indicating the condition of the strands and grout inside the anchor. The field logs of the borescope inspections are included in Appendix B of this report. Video results of the borescope inspections are stored at the FDOT District 3 Maintenance Office.

Figure 2.4 shows photographs of borescope inspections of different anchorages in the Mid-Bay Bridge. These photographs show four typical conditions found during inspections of the anchors.

Figure 2.3 – Borescope Inspection of a Post-Tensioning Tendon Anchor.

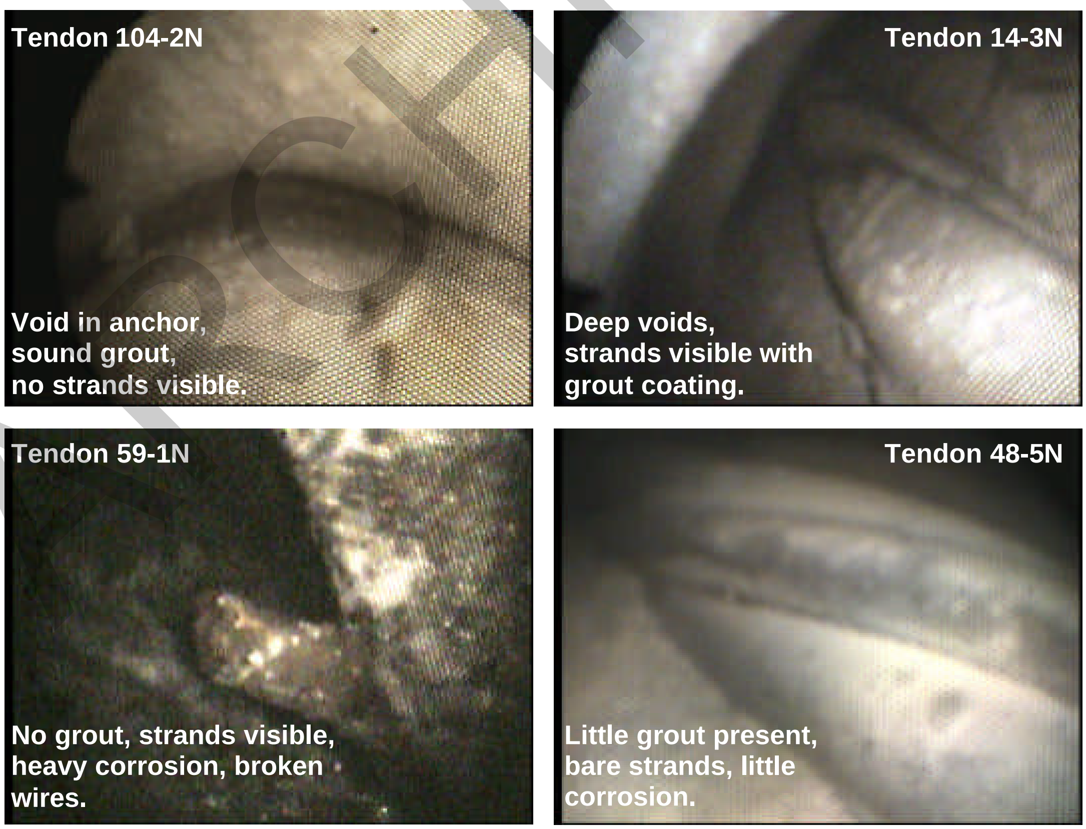


Figure 2.4 – Still photographic results of Borescope Inspections.

2.4 Vibration Testing

Vibration testing of the post-tensioning tendons was conducted by Dr. A. A. Sagues, P.E., Ph.D., of the University of South Florida. Vibration testing was first performed on the remaining five tendons of both Spans 28 and 57 during the evening of August 28, 2000 and early morning of August 29, 2000. The results of this testing gave the FDOT sufficient confidence to re-open the bridge to two-axle vehicular traffic. Borescope inspections (Section 2.3) of the anchorages of these tendons provided the FDOT with additional information based upon which they subsequently disallowed vibration testing as the sole source of evaluation (See discussion in the last paragraph of this Section).

Complete vibration testing of all tendons was conducted from October 2, 2000 to October 9, 2000. This testing consisted of measuring the vibrational response of tendons to mechanical excitation, and using the results to estimate forces in the tendons. Comparison of results for the various tendons in the bridge can be used as a possible indicator that a tendon may be in distress. Figure 2.5 shows photographs of the vibration testing and the visual display of a tendon test.

The vibration testing begins by manually striking the tendons with a hammer, and recording the resulting vibrations for later analysis. A "dead-blow" hammer was used, striking perpendicular to the tendon axis. The head of this type of hammer contains metallic shot in a yielding plastic enclosure, thereby minimizing damage to the polyethylene tendon duct and reducing the chances for multiple bouncing impacts.

Each tendon was tested in each of its three free lengths: from the south diaphragm to the deviation beam (Zone A), from deviation beam to deviation beam (Zone B), and from deviation beam to north diaphragm (Zone C). The impact point was at a distance of 1/6 of the free length from the end of the zone being tested. A single axis accelerometer was attached temporarily with wax to the polyethylene duct at a point distance of 1/3 of the free length from the end of the zone being tested. The accelerometer axis was normally parallel to the direction of the hammer blow, so that in-plane vibrational modes would be detected.

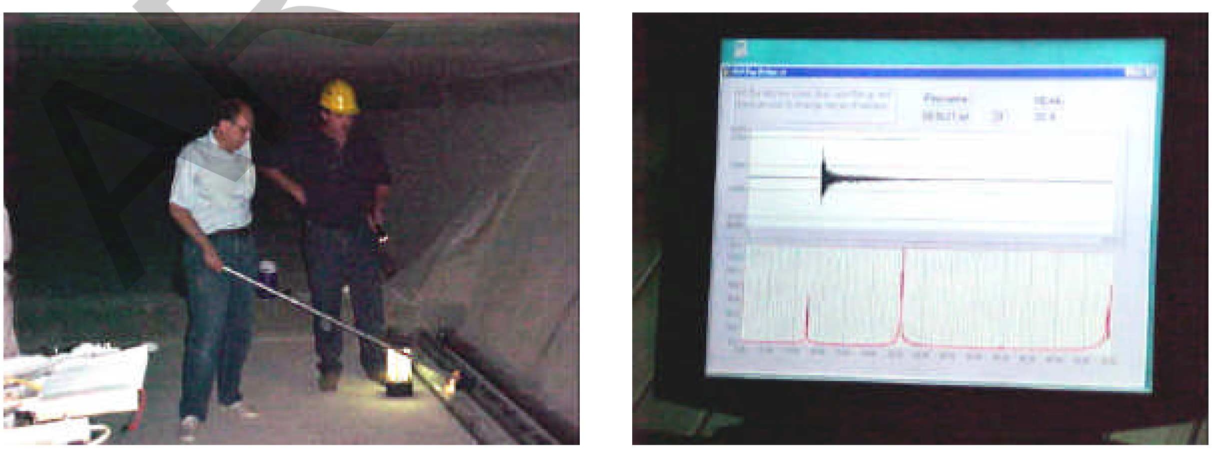


Figure 2.5 – Vibration Testing

Signal recording was performed using a laptop computer and proprietary software to acquire stereo audio input. The software creates an audio file (*.wav) of the recording that was stored in the computer hard drive, and provides visual indication of the waveform and spectral distribution obtained, allowing for immediate feedback in case a test needed to be repeated. Another proprietary computer program was used to compute the tension in the tendon.

The raw data, synthesized results, and Final Report of the vibration testing for the Mid-Bay Bridge post-tensioning tendons is presented in Appendix C of this report. Figure 2.6 shows a summary plot comparing the stresses in Zone A and Zone C of the tendons. Equal forces in the two zones would result in a point plotted on the 45° line. The variations of all of the tendons are essentially within a +/-6% variation from equal values, indicating no significant loss in forces along the length of the tendon. Some variation is expected to exist as a result of friction developed during stressing of the tendons.

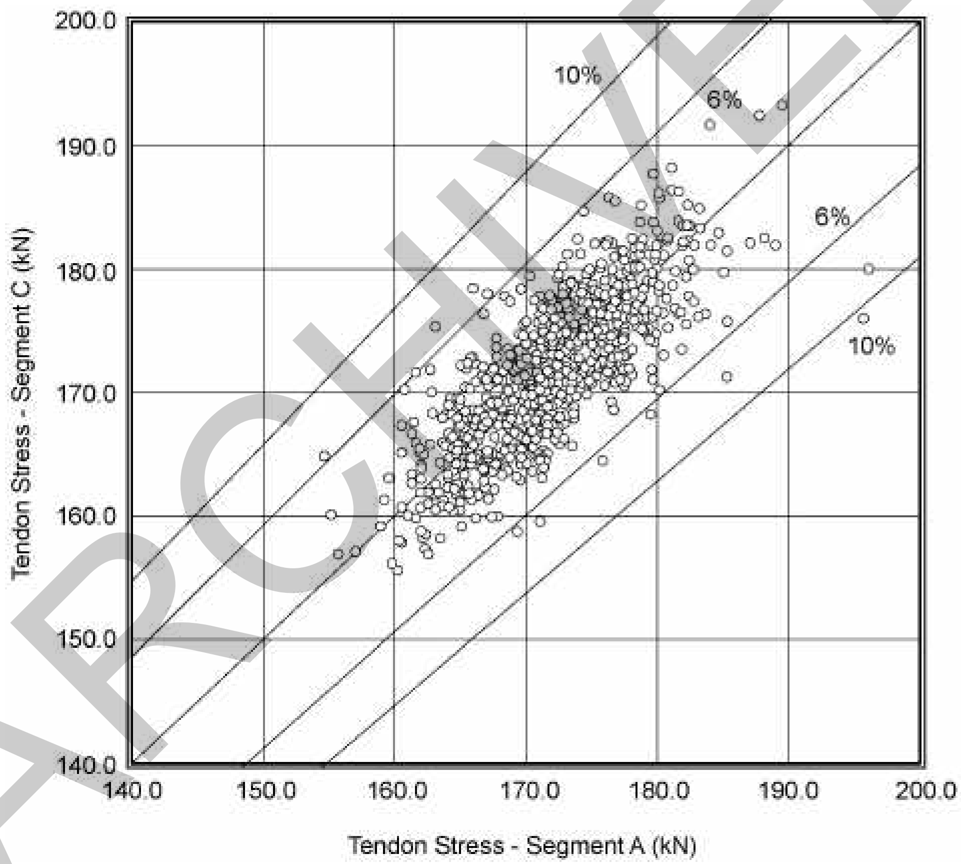


Figure 2.6 – Comparative tendons forces from vibration testing.

The vibration testing of Tendon 9-1 gave an indication that force had been lost in one portion of the tendon. Figure 2.7 shows the results for all tendons in Span 9. This bar chart shows a drop in the force in Tendon 9-1 between the deviation diaphragm and the expansion joint pier segment. The borescope inspection of this anchor showed heavy corrosion and wire breaks at this end of the tendon.

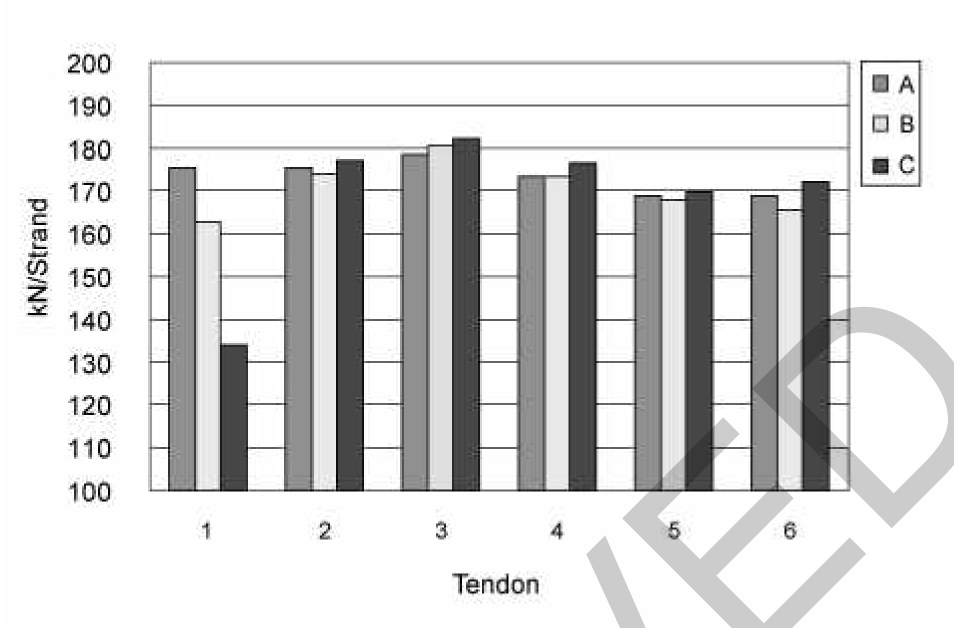


Figure 2.7 – Vibration results for Span 9

Vibration testing of post-tensioning tendons has distinct limitations. The results may not be valid for the entire length of tendon if grout in the pier segment duct and/or trumpet bonds the tendon significantly to the surrounding concrete. A positive vibration result may give an impression of a satisfactory tendon without knowledge of strand conditions in the anchor. The analytical assumptions used in data reduction are somewhat subjective and may be misinterpreted if variations in end fixity, duct condition, and grout mass are not taken into account properly. Absolute values of this type of vibration testing should be used cautiously. The results of vibration testing, as shown in Figure 2.7, are best used by comparing relative differences between Segments A, B, and C.

2.5 Mag-Flux Testing

Mag-Flux testing of the post-tensioning tendons of the Mid-Bay Bridge was conducted by Dr. A Ghorbanpoor, P.E., Ph.D., of the University of Wisconsin-Milwaukee. The on-site testing was performed from October 27, 2000 to November 2, 2000. Mag-Flux testing uses magnetic flux leakage principles to give a non-destructive evaluation of section loss of post-tensioning tendons. In this method, a magnetic field is induced around a post-tensioning tendon. Changes measured in the magnetic field can be correlated, based on previously performed calibrations, to steel section loss due to corrosion or wire breakage.

The equipment used in Mag-Flux testing consists of a mechanical frame that supports a pair of strong permanent magnets and a series of magnetic field sensors. Data received from the magnet/sensor assembly is collected by data acquisition software on a laptop computer to facilitate data recording, displaying and interpretation. The magnet/sensor assembly rides along the free length of the external post-tensioning tendons on a set of contact wheels. Contact wheels attempt to maintain a constant distance of 0.25 inches between the face of the magnet/sensor assembly and the surface of the polyethylene duct of the post-tensioning tendon.

Testing of the post-tensioning tendons consisted of placing the magnet/sensors assembly of the test machine on the three free length zones of each tendon (Zone A, B and C). The magnet/sensor assembly was moved with a steady motion along the tendon. Magnetic flux leakage data from the tests were transmitted to a computer and recorded. The synthesized data was displayed back to the investigator in the form of real time plots for the different sensors on the test machine. The investigator was able to evaluate the real time plot and note the location along the length of the tendon where section loss from corrosion or wire breakage occurred. Figure 2.9 shows the max-flux testing operation.

Figure 2.9 – Mag-Flux Testing

Mag-Flux Testing at the Mid-Bay Bridge resulted in identifying two locations on the post-tensioning tendons where corrosion and section loss had occurred. These locations and the findings were:

- A positive test result for Tendon 71-1 indicated a possible section loss in Zone A of the tendon, 30' from the start of the test. Physical examination at this location found a small hole present in the polyethylene duct. A small window was cut in the duct to further determine the extent of the corrosion. Four wires of a seven-wire strand were heavily corroded just below the surface of the hole in the duct (See photograph in Appendix D). This level of corrosion was not considered severe enough to warrant replacement of the tendon. The duct was sealed following inspection.

- A positive test result for Tendon 98-5 indicated a possible section loss in Zone C of the tendon, from 9' to 10.5' from the start of the test. Physical examination of this location found another small hole in the polyethylene duct. A small window was cut at this location of the tendon and heavy corrosion limited to the four wires of a seven-wire strand was found (See photograph in Appendix D). This level of corrosion was not considered severe enough to warrant replacement of the tendon. The duct was sealed following inspection.

Mag-Flux testing of external post-tensioning tendons is limited in the ability to predict the condition of post-tensioning tendons. The testing procedure is only able to establish the location in a tendon where section loss has occurred. The amount of section loss is difficult to estimate without physical examination of the tendon. Mag-Flux testing is very sensitive to variations in the position of the magnet/sensor assembly relative to the tendon. Imperfections on the surface of the tendon or previous repairs in the form of wrappings can change the clear distance from test equipment to the prestressing steel and produce poor results. In addition, Mag-Flux testing cannot predict the presence of voids in the grout or bleed water pockets. As a result, Mag-Flux testing cannot be used to find possible locations of future corrosion resulting from lack of protection by grout.

A secondary benefit of Mag-Flux testing is locating small imperfections in the ducts of external tendons. Though the testing does not measure any results with regard to the duct, typically duct imperfections coincide with prestressing corrosion at the same location. These duct defects can be sealed and further corrosion at this location arrested.

This correlation of tendon corrosion and duct defect focused attention on a construction and inspection practice that led to unnecessary corrosion of external post-tensioning tendons. Often, during grouting, the tendons were sounded to determine if there were voids in the grout. When a probable void was found, a small nail was tapped into the duct to verify the presence of grout. Holes left open after this inspection provided a localized point of entry for warm, humid air and the opportunity for localized corrosion of the tendons. This same type of corrosion cell has been found in other post-tensioned bridges in Florida and may have been the cause the failure of Tendon 28-6 (Section 3.1). The dry joints on this bridge have efflorescence which is evidence of slight water leakage that may be contributing to the humidity which can fuel these slow growing corrosion cells.

In general, the number of tendon corrosion locations detected by Mag-Flux testing, was very low considering the number of tendons in the bridge. However, many of the ducts that had already been sealed by heat wrapping could not be successfully tested for reasons stated above. The complete report of the Mag-Flux testing program is provided in Appendix D of this report.

2.6 Visual Tendon Inspections

Visual Tendon Inspections were made at locations in the free lengths of the post-tensioning tendons of the Mid-Bay Bridge. These locations were identified during the initial "walk-through" inspections, sounding inspections, and mag-flux testing. The visual inspections were performed by removing portions of the polyethylene ducts from tendons that exhibited indications of corrosion. With the duct removed, the grout and strand could be inspected. Both active and inactive corrosion were found at several of the locations inspected.

Figure 2.8 shows a stripped portion of Tendon 40-2. The grout in this portion of Tendon 40-2 was very fractured and chalky. Two wires in one of the seven-wire strands were broken. Two other wires in the same strand broke as additional grout was removed from the section. This portion of the tendon was marked for future patching and wrapping. Results of the Visual Tendon Inspections are provided in Appendix E of this Report.

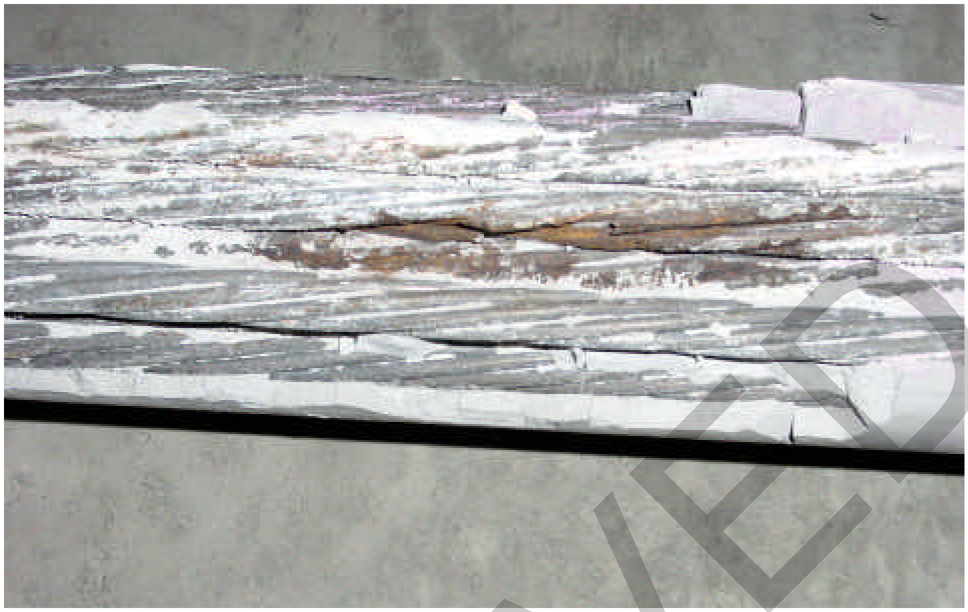


Figure 2.8 – Visual Inspection of Tendon 40-2 with corrosion in the free length of tendon.

2.7 Mock-Up and Field Trial For Filling Voids In Anchorages

A significant task of the rehabilitation of the post-tensioning system of the Mid-Bay Bridge was filling voids inside the anchors with grout. Several grouts and methods of injection were investigated, with the goal of finding a system that would consistently fill voids completely. The grout chosen for the repair was Master Builders 816 Cable Grout. This cementitious grout meets the requirements of the interim grouting specifications currently in use by the FDOT. Details of two grout injection methods were developed. These two methods were:

- Pressure Injection - Placement of grout under positive pressure through a straw inserted through the grout port, deep into the void and then retracted as the void is filled.

- Vacuum Injection - Placement of the grout under a vacuum produced by drawing the air out of the void. The negative pressure is used to draw the grout into the anchor.

Mock-up testing using the two proposed methods of grout injection was conducted to determine which produced the most effective repair of the anchor. The mock-up tests were performed using the same components of the post-tensioning system used in the bridge. Post-tensioning strands were placed inside the anchor and a length of duct and then partially filled with grout. The mock-up tests were slightly inclined to replicate voided conditions similar to those found during the borescope inspections. Figure 2.10 shows a sketch of the mock-up test specimen.

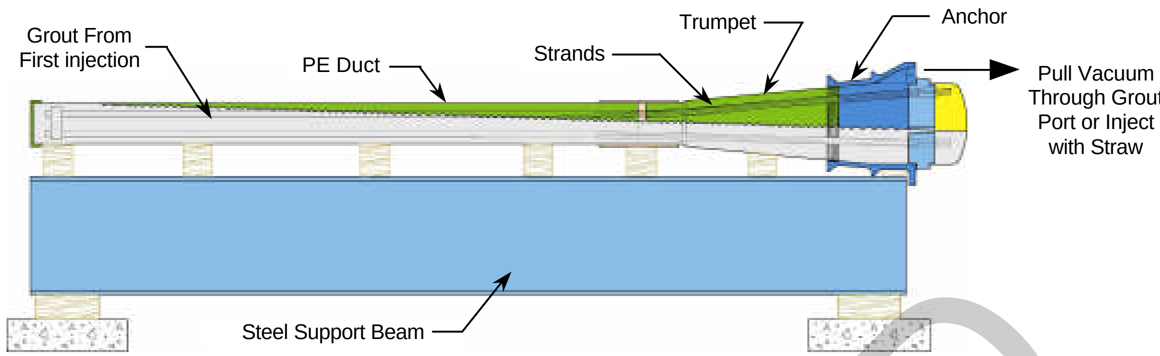


Figure 2.10 – Mock-up test specimen

After the first grout placed in the test specimen hardened, the two different methods of injection, positive pressure and vacuum injection, were used to fill the voids in the anchors. Figure 2.11 shows the two injection methods in progress. The positive pressure method is shown on the left with the straw inserted into the grout port of the anchor and a hand pump being used to force the grout into the voids. The vacuum injection method is shown at the right in Figure 2.11. One end of the grout injection tube is connected to the grout port. The other end is attached to a switchable manifold. The air is first removed under vacuum, being drawn out by the grout-metering pump. The volume is measured on the vacuum and therefore provides an estimate and degree of confidence in the amount of grout to be placed. Generally the amount of grout placed in the Mid-Bay Bridge was slightly higher in volume than what the vacuum indicated. In the second stage of work the manifold controls are switched, causing the vacuum to pull the grout into the void.

Figure 2.11 – Injection methods: Positive Pressure (left), Vacuum Injection (right).

Following the injection and hardening of the secondary grout, the test specimens were cut into sections along the length of the tendon. These sections were reviewed to assess the effectiveness of the grout injection methods. Figure 2.12 shows a section of specimen injected under positive pressure on the left, and a section of a specimen injected under a vacuum on the right. Visual inspection of the autopsied tests indicated that the vacuum injection method filled the voids in the mock-up samples better than the positive pressure method. The specimens injected under a positive pressure still had large voids, as shown at the left in Figure 2.12. Voids were not present in the tendons injected by vacuum and there was a consistent flow of the grout into the voids and small annular spaces between the primary grout and duct (note the extent of the lighter colored grout at the top of Figure 2.12 on the right).

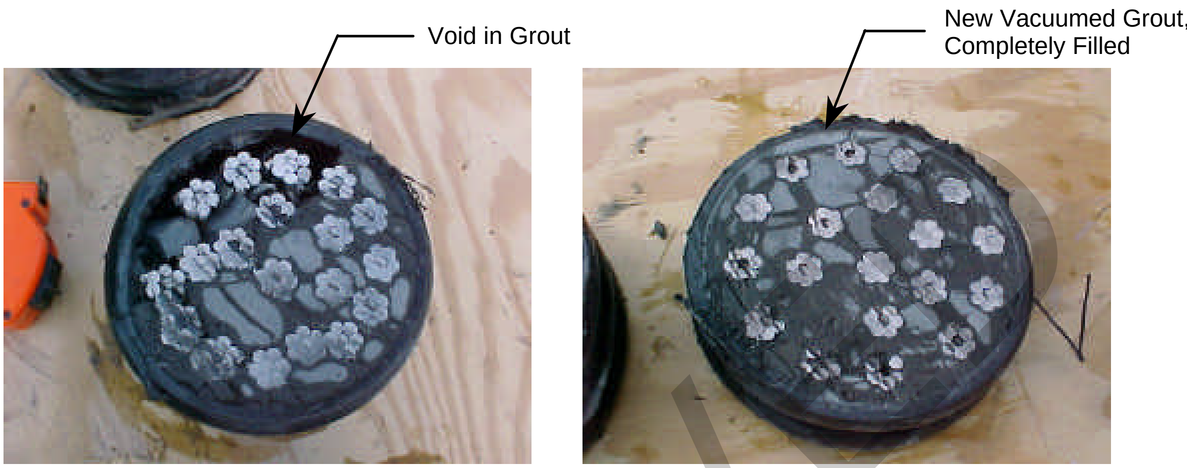


Figure 2.12 – Mock-up tests segments: pressure injection (left) and vacuum Injection (right).

Following the mock-up testing, a field trial was conducted to determine procedures for using the vacuum method of injection in the rehabilitation of the Mid-Bay Bridge. The photographs in Figure 2.13 show different aspects of the field trial. The photograph at the bottom right of Figure 2.13 shows the finished grout cap for this tendon. Based on the mock-up testing and field trial, the vacuum method of injection was selected for filling the voids in the anchors of the Mid-Bay Bridge.

Figure 2.13 – Field trial of the vacuum injection method.

2.8 Other Related Testing

Several other tests were performed on the components of the post-tensioning system in order to evaluate their impact on providing corrosion protection. The results of these tests are presented in this section. The complete reports are provided in Appendix F of this report.

2.8.1 Polyethylene Duct Testing

Two independent testing laboratories were asked to test characteristics of the polyethylene duct used on the Mid-Bay Bridge. A characteristic of polyethylene is that samples can be melted and reconstituted into testing samples while maintaining their physical properties.

Hancor, Inc. (Hancor) evaluated the duct for conformance to ASTM D 3350 cell class 345433C. This was the ASTM testing procedure and cell class specified in the project construction specifications. Four of the tested characteristics (density, melt index, flex modulus and tensile strength) met the ASTM requirements. The duct material did not meet the test requirements for the Environmental Stress Crack Resistance (ESCR). The ASTM requirement specifies that not more than a 20% specimen failure is permitted over the 192 hour test duration. The tests performed by Hancor for the duct for the Mid-Bay Bridge exhibited 100% failure to the test in less than 24 hours.

Atofina Petrochemicals, Inc. (Atofina) performed chemical and rheological testing on the polyethylene duct of the Mid-Bay Bridge. The chemical testing indicated that the percent of carbon black in the post-tensioning ducts was 1.2% and that the density of the material was higher than anticipated based on chemical composition. The ASTM requirement for carbon black content in this material is a minimum of 2%. The rheological tests indicated that the base resin was a medium molecular weight grade lower than commonly used in pressurized pipes. The Atofina tests state that the combination of high density and low molecular weight produces a product of high brittleness.

These two brief reports, included in Appendix F, indicate that the polyethylene ducts did not meet the requirements of the project construction specifications, and could be a source of duct cracking.

2.8.2 Grout Testing

The FDOT State Materials Office in Gainesville, Florida performed tests to evaluate the chemistry of the grout. A sample of the results of one of these tests for the grout in Tendon 40-2 is provided in Appendix F of this report. The FDOT State Materials Office consistently found the pH values of the grout to be appropriate and the chloride content to be on the order of 0.25 pounds of chloride per cubic yard of grout. This chloride content is a trace amount consistent with expected values for cement-based materials.

SKW/MBT, Inc. (SKW/MBT) of Cleveland, Ohio performed chemical and petrographic examinations of grout samples. Specifically, SKW/MBT investigated whether grout expansion could have caused the cracking of the polyethylene duct. The results of the tests by SKW/MBT did not indicate any unusual conditions that would have produced unexpected expansion. The testing indicated that the water cement ratio of the grout was very high and that grout characteristics varied with depth from the surfaces formed by the ducts inward towards the strands. This variation in grout characteristics indicated a significant migration of bleed water. (See Appendix F)

2.8.3 Prestressing Steel Testing

Samples of the corroded prestressing strand of Tendon 57-2 were tested for tensile strength by the FDOT Structures Research Center. The results of these tests indicated that strand pitted to the extent of this tendon had a reduction in ultimate strength of 12%. This reduction in strength was determined by comparing test results for corroded portions of the strands to tests on portions of the strands that were not corroded. (See Appendix F)

The level of corrosion on the portion of Tendon 57-2 that was tested was representative of several locations on several tendons in the bridge. This level of corrosion was not, however, the most severe found in the bridge. Limitations of the testing equipment did not permit the axial testing of the most heavily corroded areas on the strands. These more severely corroded locations occurred near the ends of the tendon where the voids in the anchors exposed prestressing strands to high corrosion. There was not enough length of strand available on either side of the heavily corroded area for the testing equipment to grip.

2.8.4 Tendon Potential Testing

Filling voids in anchors with secondary, vacuum injected grout created concern that the water introduced with the new grout could further aggravate active, or re-activate corrosion of the prestressing steel. To study the effect of the new grout on existing tendon corrosion, the FDOT Materials Laboratory developed an in-situ testing procedure to measure the change in electrical potential within the new grout as it cures.

Borescope records of the Mid-Bay Bridge post-tensioning tendons were reviewed and six tendons with various levels of corrosion were selected for testing. In addition, one of the newly installed replacement tendons was grouted during the tests and used as a control specimen. One anchor of each of the six tendons was vacuum injected with secondary grout. A small hole was then drilled into the grout through the bleed hole in the anchor plate of the six re-grouted tendons and the new control tendon. A wooden dowel saturated in a three percent sodium chloride solution was inserted into the hole and placed in contact with the new grout. Copper-copper sulfate electrodes were place in contact with the dowel and potential measurements taken. The tests were performed for eleven weeks.

Figure 2.14 shows a plot of the variation in electrical potential within the grout over the eleven-week test period. Test results indicated that potentials reached a passive state, as defined by ASTM standards, shortly after re-grouting. Also, the potentials of the re-grouted tendons were similar to those of the newly grouted replacement tendon. Based on these findings, secondary vacuum grouting of tendons was incorporated into the repair procedures of the post-tensioning tendons. Complete results of these tendon potential tests are provided in Appendix F of this report.

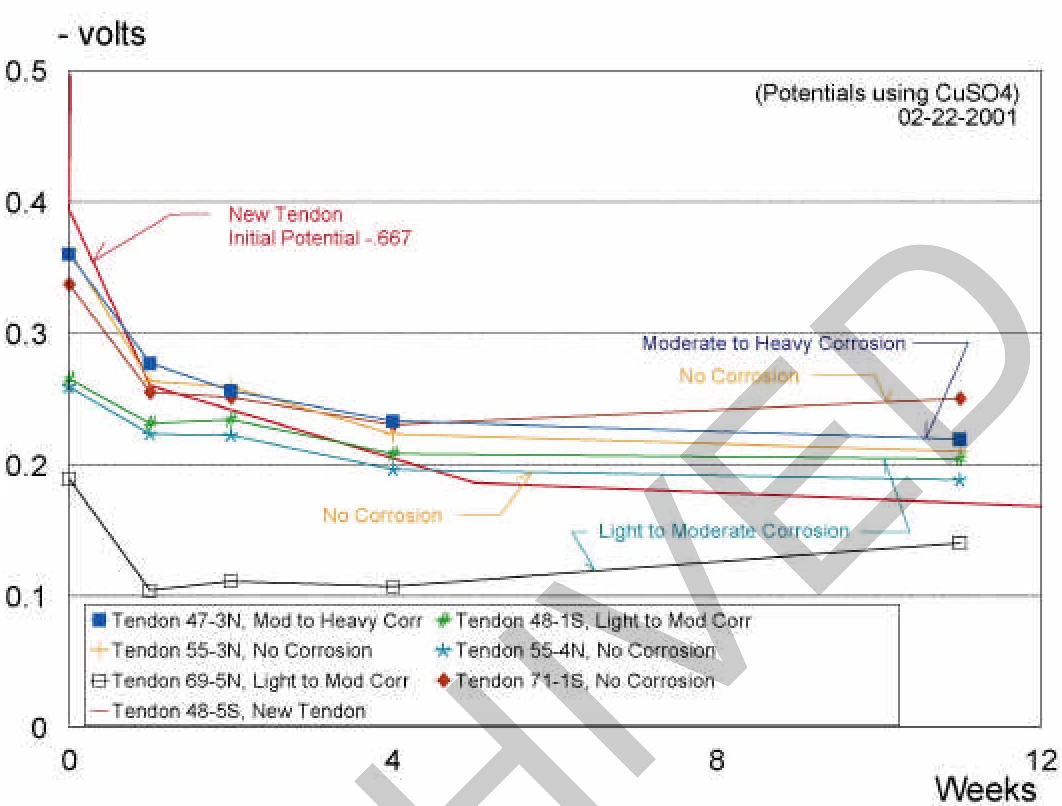


Figure 2.14 – Variation in electrical potential with time.

Chapter 3 – Findings Of The Inspection And Testing Program

3.1 Failed Post-Tensioning Tendons

Tendon 28–6 was discovered partially failed on August 28, 2000 during a regularly scheduled FDOT annual inspection of the Mid-Bay Bridge. The inspectors found the duct at this location to be significantly cracked and bulging. Figure 3.1 shows the corrosion of Tendon 28-6 after the duct had been cut away in the free length of the tendon between a deviation diaphragm and expansion joint pier segment diaphragm. The extensive damage to the duct did not permit the evaluation of whether defects in the duct prior to strand failure may have led localized corrosion of the tendon.

Figure 3.1 – Tendon 28-6

An immediate “walk-through” inspection of the Mid-Bay Bridge was conducted after finding the damage to Tendon 28-6. This inspection found that Tendon 57-1 had completely failed. This was evidenced by the complete pull out of the tendon and embedded steel duct from the expansion joint diaphragm. The failure of Tendon 57-1 is shown in Figure 3.2.

Figure 3.2 – Tendon 57-1: failure at Expansion Joint Diaphragm (left),
View towards Pier Segment Diaphragm (right)

The corrosion of the prestressing steel in the anchor of Tendon 57-1 produced a loss in cross sectional area, which led to strand failures. The force from these failed strands was then restrained through bond with the steel duct through grout that was present in the duct. Load in the steel pipe was transferred to the surrounding concrete. Forces in the remaining unbroken strands were carried back to the anchor plate. Eventually the corrosion and breaking of additional strands was sufficient to exceed any resistance provided by the remaining prestressing strands causing all the load to be transferred to the steel duct/concrete interface. Bond between the rigid duct and the diaphragm concrete then failed allowing the tendon to slip into the span.

Post-mortem inspection of the strands in the anchor plate of Tendon 57-1 revealed that the less corroded wires had necked-down fractures, indicating sudden failure and transfer of load to the steel duct. This was consistent with the failure mode described above.

Post-tensioning anchorages are designed to transfer prestressing loads from the strand, through the wedges, to the anchor plate, and then through the bearing surfaces of the anchorage to the concrete. Though not considered in the design of the anchorage itself, the analysis, testing and approval of post-tensioning systems all consider complete grouting. It is clear that the tendons of the Mid-Bay Bridge were not constructed with grout to the same details as used during the development and approval of the post-tensioning system. The consequence of the difference will be investigated further by FDOT through a research project.

Figure 3.3 shows two other photographs related to the failure of Tendon 57-1. The photograph on the left shows the anchor plate after removal from the anchorage. The majority of the strands are still held in the anchor plate by the wedges. The extent of the corrosion and the nature of the corrosion-induced breaks are evident. The photograph on the right of Figure 3.3 shows the extent of the corrosion inside of the multi-plane anchor casting. This photograph also gives evidence that the water contributing to corrosion was sealed in place in the anchor during construction. The protective epoxy coating and black mastic seal was completely intact at the time of the discovery of the failed tendon. The light gray circumferential break, shown completely around the anchor, is the epoxy layer of the anchor protective coating.

Figure 3.3 – Anchor plate and anchor of Tendon 57-1

Tendon 28-6 and Tendon 57-1 were the only tendons that had experienced failure. Each one had failed because of a loss in cross sectional area due to corrosion. Tendon 57-1 failed in the anchor just behind the anchor plate. Tendon 28-6 failed in the free length of tendon. Both of the tendons were located in expansion joint spans and both were the tendons in the uppermost positions. Later inspections confirmed the majority of concentrated voids and corrosion occurred in the most highly draped tendons.

3.2 Voids In Post-Tensioning Anchors

Review of the failure of Tendon 57-1 showed little or no grout in the anchorage immediately behind the anchor plate at the expansion joint end of the span. Based on this information, the other ten anchors of this span were inspected by borescope as described in Section 2.3 in this report. Each of the anchorages inspected at this time showed significant voids in the anchors and corroded strands.

Engineers are aware that small voids can occur in post-tensioned concrete construction. The effect of a void on tendon durability is influenced by factors such as: exposure and condition of strands, condition of grout, possible paths of recharging (defined below), relative structural significance of the tendon, and surrounding environment. Voids that leave the strands susceptible to attack from corrosion are not acceptable and require filling with grout. The Florida Department of Transportation, considering factors similar to those just listed, used engineering judgment to decide that no known void in an anchorage would be acceptable in the Mid-Bay Bridge. All voids would be filled with grout using the vacuum injection techniques described in Chapter 2.

The presence of the voids and corrosion in the post-tensioning tendons most likely resulted from one, or a combination of, the following items:

- Contamination – Water, with or without corrosive aggravating elements, penetrating an ungrouted anchor or duct before or after tendon stressing. The source of this water could have been from deck runoff, atmospheric humidity, or from flushing of the ducts. Salt spray may also have carried chlorides into the duct system.
- Leakage of grout – “Blow outs” of the grout at the neoprene boot connections between steel and polyethylene could have led to a loss of grout.
- Bleed water – Excessive free water in the grout moving to tendon high points allowed the remaining grout to settle back into the ducts away from the anchors. This can be aggravated by excessive water in grout.
- Subsidence of the grout – Grout in tendons that are filled from a high point can cavitate and capture air in the grout column. Before the initial setting of the grout the captured air, possibly combined with gas from expansive agents, rises to the high point, in this case inside the anchors behind the anchor plates.
- Settlement – gravity induced separation of cement from the water in the grout mix.
- Recharge – Deck runoff may flow over the anchorages after grouting and before anchor protection is applied and expansion joints are installed. The exposed porous grout without pour-back and mastic protection in place may have absorbed water. This phenomenon has been documented at another Florida post-tensioned bridge anchorage in a vertical application. Another potential for recharge and continued strand corrosion is through a separation between a pour-back and bulkhead. Most anchorages on the Mid-Bay Bridge did not have a pour-back and therefore the void may have been exposed to recharge prior to the re-grouting of the cap and/or mastic installation.

It is important to note that the most significant voids in the ducts of the Mid-Bay Bridge were at the expansion joint diaphragms. Given an amount of bleed water and cavitation, the total volume of the void at either end of the tendon should be nearly the same. At interior pier segments this volume would be distributed over the horizontal tendon profile resulting in long but thin voids that would not expose strands. At expansion joint pier segments the voids would collect at the tendon high point just behind the anchor plate and would be shorter and deeper, exposing strand. The change in inclination of the expansion joint span tendons as they anchor at the expansion joint diaphragms would produce deeper voids in tendons 1 and 6 and shallower voids in tendons 3 and 4. All of these tendencies were consistently found in the Mid-Bay Bridge, as revealed by the borescope inspections.

3.3 Grout Quality

The general impression held by the inspectors and investigators involved with the review of the Mid-Bay Bridge is that the grout is of suspect quality in many areas. Samples taken and tested were described as soft, chalky and visibly porous. The nature of the components of the grout along with admixtures was chemically analyzed for compliance with project construction specifications. The water/cement ratio and expansive agent content of this grout may not have been correct. The petrographic investigations found that the grout was poorly mixed, in that un-hydrated cement particles were found near the outer perimeter of the tendons. The grout allowed the excess bleed water to migrate upward through the grout near the strands. This characteristic is consistent with inspection findings where water migrated upward into the tendon anchorages, thus creating or adding to the voids in the anchorages.

Color can be another indication of grout quality. The usual color of grout used in this post-tensioning application is light gray. Much of the grout in the tendons requiring replacement in the Mid-Bay Bridge was white, indicating high water content and grout segregation. The significant variation in color of the grout from the top of the duct (white) to the bottom of the duct (darker gray) was further indication of a higher than normal water content. Much of the pour-back grout in the grout caps has a dark gray color with visible silica. This grout was either poured or packed into the grout caps and did not flow into the trumpet area through the wedge plate. The shortcoming of this secondary placement of grout over the anchor plate is that it does not flow back into the anchor, filling voids that may be present.

Figure 3.4 shows two examples of dark gray secondary grout in anchors of the Mid-Bay Bridge. The photograph on the left shows the dark gray grout used as a pour-back to fill the cap during construction, leaving in place a partially voided trumpet. The light colored grout in this photograph comes through the grout bleed hole into the cap during grouting. Caps with subsidence or partial fill were consistently found to be indicative of a partially filled trumpet. The photograph on the right shows the dark gray grout throughout the anchor.

Figure 3.4 – Secondary Grout in Anchor Caps

3.4 Cracking Of Polyethylene Duct

The polyethylene ducts are substantial cracked throughout the Mid-Bay Bridge. Duct cracking was not a new issue discovered in association with recent inspection of the bridge. Cracking of ducts had been observed since the bridge was opened and protective wrappings have been applied in a previous maintenance contract (1997). More extensive cracking has occurred since these first maintenance operations were conducted.

The polyethylene duct was tested to see if the characteristics of the duct were appropriate for this application. The results of these tests indicate that the duct did not meet the requirements of ASTM D 3550 and cell classification as specified in the project construction specifications. Further testing indicated that the high density and medium molecular weight grade of the resin produce a brittle polyethylene that perhaps should not have been used in this application. Radial stresses induced during grouting would contribute to the poor performance of the ducts.

3.5 Protection of Post-Tensioning Anchorages

Visual and random sounding inspections of the protective coatings of the anchorages were conducted. The visual inspection identified cracking and/or spalling of the coal tar epoxy coating. These findings would indicate that some locations of failed protective coatings are the result of the expansion of the steel beneath the coatings caused by corrosion induced by the wicking of water trapped in the void.

Striking the grout caps produced a hollow sound at many locations where there was no visible external damage to the protective coatings. Removal of these intact protective coatings typically revealed a mix of white and sandy gray grout in the grout cap. Locations with hollow sounding, intact protective coatings also had voids in anchors and corrosion on the strands and anchor plates.

3.6 Corrosion Of Prestressing Steel

As a result of the deficiencies in the post-tensioning system of the Mid-Bay Bridge mentioned above, there has been considerable corrosion of the prestressing steel. The following list is a compilation of the features, tendencies, or practices that most likely affected the accelerated corrosion of the tendons.

- The majority of the corrosion occurs in the tendon anchorages at the expansion joint segments. The tendon geometry described in Section 1.2 led to more concentrated voids and more strands exposed to the moisture in the anchorages.
- The most serious corrosion occurs south of the main span. Project correspondence indicates the approval of the use of an anti-bleed grout mixture on 10/19/92 for future grouting use. Construction records indicate that construction had progressed to Span 69 by the day this letter was issued.
- Water inside the anchors where corrosion was found most likely came from excessive bleed water and recharging during construction. However, general atmospheric attack cannot be ruled out. Construction records reviewed did not note the dates that grout cap pour-backs were made or anchor protective coatings applied. Records did show that

expansion joint assemblies were not placed immediately following grouting of the particular expansion joint span. The concrete pour that secures the expansion joint also provides protection to the expansion joint span anchorages from rain and deck runoff. It does appear, however, that the moisture causing corrosion was introduced during construction, because the protective coatings were found intact at anchors where strand corrosion was found.

- Single-end grouting from the high point most likely entrapped air (cavitation) and subsequently the grout subsided from the anchorages. Grouting rates may have been too high, resulting in turbulent flow, and contributing to the amount of entrapped air and bleed water.

- The protection offered by the polyethylene ducts was compromised by punching small holes in the ducts while inspecting the ducts during construction.

- The protection offered by the polyethylene ducts was compromised as the result of extensive cracking in the ducts since the completion of construction.

- The high permeability of the grout offers less than expected protection to the prestressing strands. This is even more pronounced when the polyethylene duct is damaged.

- The time interval between removal of post-tensioning steel caps and application of protective coatings may have allowed the recharge of moisture.

- Although the corrosion found in the vicinity of the anchorage assembly was believed to be primarily caused by the presence of grout voids and grout bleed water, this corrosion activity may also have been aggravated by galvanic corrosion between two or more dissimilar metals that make up the post-tensioning system. There are at least six different metals in the immediate vicinity of the anchorage assembly (strands, chucks, wedge plate, trumpet, duct pipe, zinc) Except for the zinc, these metals are very close on the electromotive series (under standard conditions) and would not be expected to have significant potential differences. Therefore these metals would not be expected to corrode when coupled and surrounded by cured, cement-based grout of reasonable quality. The zinc layer would be expected to be galvanically active during the period of time beginning with the introduction of grout and continue briefly until the grout has cured and developed high electrical resistance. No significant corrosion of the system would be expected for many decades unless the system was to be breeched such that water and oxygen and/or contaminants were allowed to enter the system.

  In instances where the trumpets contain voids and water, corrosion of one or more metals is almost certain to take place. Where the void is sufficiently extensive to involve the galvanized pipe, it would be expected that, at least initially, all of the other metal components in contact with the electrolyte (water and wet grout) would benefit by some degree of cathodic protection because of the highly anodic potential of the zinc and its propensity for rapid dissolution in high pH media such as that which would initially be found in grout bleed-water. The efficiency of the zinc in providing effective cathodic protection to the other metallic components is highly dependent on numerous factors such as solution chemistry and resistivity, oxygen availability and polarization characteristics of both the zinc and the other metals in electrolytic contact with one another. Likewise, in the absence of the zinc providing a protective function, the other metal components would corrode dependent upon the very same factors.

The actual potential of the individual metals would dictate whether one or more metals would corrode preferentially to another. In this instance, the solution chemistry and oxygen availability would play a significant role in the development of metal potentials and resulting galvanic corrosion rate. Therefore the possibility of galvanic activity between the various metals in the anchorage assembly cannot be ruled out. This is particularly so since it has been clearly shown that prestressed strands are particularly susceptible to corrosion when exposed to grout bleed water. For example, reliable studies (References 1 and 2 listed below) have shown the propensity for extremely high corrosion rates for prestressed strands when exposed to grout bleed water. In fact, Reference 2 demonstrated total tendon failure due to corrosion from grout bleed water in just a matter of weeks. Reference 1 demonstrates a particularly high propensity to bleed water development and subsequent strand corrosion when Sika's Interplast N admixture (as used at the Mid-Bay bridge) is used in ordinary grout.

References:

1. *"Performance of Grouts for Post-Tensioned Bridge Structures",* Publication No. FHWA-096, Federal Highway Administration, Washington, D.C., December, 1993*RD-92-095.*

2. *"Implications of Test Results from Full-Scale Fatigue Tests of Stay Cables Composed of Seven- Wire Prestressing Strand",* Habib Tabatabai, A. T. Ciolko and T. J. Dickson, Reprinted from Conference Proceedings 7 of the Fourth International Bridge Conference, Volume 1, Transportation Research Board, National Research Council, Washington, D. C.

The primary reason corrosion has occurred in the post-tensioning tendons of the Mid-Bay Bridge is that water was sealed in voids in the tendons, most likely since the time of construction, in enough volume as to not be readily absorbed in the grout as it cured. This, combined with the presence of oxygen, both entrapped and/or diffused into the system over time through holes, cracks or leaks, allowed corrosion to progress. The opportunity for corrosion is enhanced by the configuration of the individual 7-wire strands. Specifically, the interstitial areas give opportunity for numerous locations for a crevice corrosion effect that is further enhanced by strand-to-strand contact within the tendon bundle. Visual observations of partially or completely failed tendons indicated that corrosion occurred over time, as there were numerous wire breaks that had continued corrosion on the broken surfaces and rounding of broken wire edges.

Chapter 4 – Post-Tensioning System Rehabilitation

4.1 Introduction

This chapter presents the remedial actions undertaken to rehabilitate the post-tensioning system of the Mid-Bay Bridge. The rehabilitation efforts are grouped into the following general categories:

- Replacement of Post-Tensioning Tendons
- Repair of Tendon Anchorages
- Duct Wrapping

Eleven post-tensioning tendons were replaced in October and November of 2000. Repairs to tendon anchorages were performed between January 1, 2001 and July 3, 2001. Some duct wrapping has performed in conjunction with the anchorage repairs. The remainder of the tendons will be wrapped under the contract that was let on August 31, 2001.

4.2 Replacement Of Post-Tensioning Tendons

Eleven post-tensioning tendons required replacement during the inspection of the Mid-Bay Bridge. A replacement criterion was established based on early inspection results and engineering judgment. Subsequent structural analyses have verified the load carrying capacity of the bridge with reduced post-tensioning levels during repairs. The following are the components of the tendon replacement criteria:

- The corrosion appears to have caused a 25% section loss of the entire tendon cross section. The 25% loss may be a combination of pitting corrosion with observed broken wires or strands.
- No two post-tensioning tendons on the same side of the box girder, in the same span could have significant section loss.
- The borescope inspections were reviewed with regard to extent of corrosion, number of strands visible, depth of the void in the grout, depth of penetration of the borescope.
- Each candidate for replacement received a callback borescope inspection.
- Two Certified Bridge Inspectors and two Professional Engineers reviewed the results of the callback inspection.

Details of the tendon removal and replacement procedure for the Mid-Bay Bridge are given in Section 4.2.12. The close proximity of the existing anchorages to the top slab and webs of the segments did not provide sufficient space to permit the use of multi-strand jacks for stressing the replacement tendons. Strands of the new tendons were slid into place one by one, using a supporting sled. The strands were placed in order from bottom to top, and then stressed using a monostrand jack from top to bottom to be sure all strands could be effectively tensioned.

The remainder of this section documents the facts about tendons that were replaced and details of the replacement procedure. The original construction documents were reviewed to understand the relationship between exposure opportunities for contamination and recharge of the voids at the expansion joints. This review was undertaken based on the observation that 10 of the 11 tendons that were replaced were in expansion joint spans.

4.2.1 Tendon 57-1

This tendon was one of two failed post-tensioning tendons that were discovered during the annual inspection on August 28, 2000. A description of this tendon is found in Section 3.1. The tendon was replaced before the borescope inspections.

Span Type: Expansion Joint Span
Date Stressed: 9/18/92
Date Grouted: 9/25/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

4.2.2 Tendon 28-6

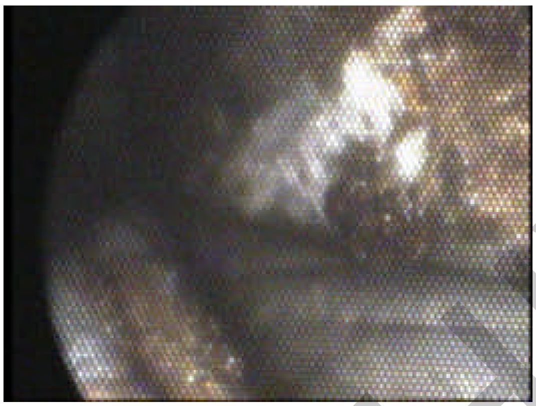
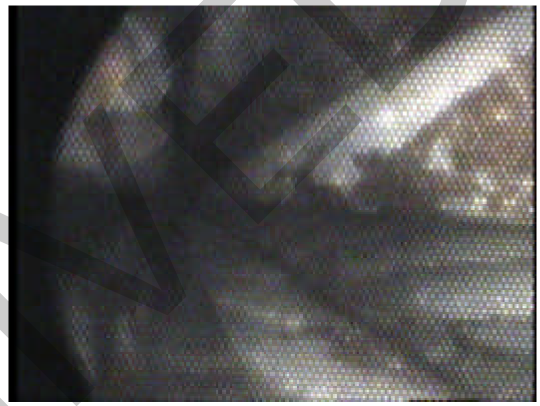
Like Tendon 57-2, this tendon was one of the post-tensioning tendons that had failed and was discovered during the annual inspection. A description of this tendon is found in Section 3.1. The tendon was replaced before the borescope inspections.

Span Type: Expansion Joint Span
Date Stressed: 7/25/92
Date Grouted: 7/28/92
Date Expansion Joint Placed: 10/21/92

4.2.3 Tendon 57-2

Span Type: Expansion Joint Span
Date Stressed: 9/18/92
Date Grouted: 9/25/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

Borescope Photographs:

Borescope Field Notes:

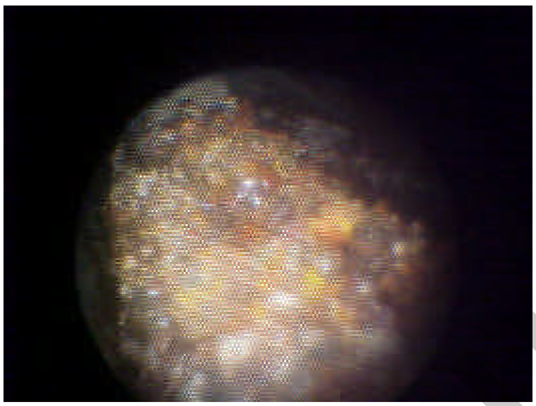
North Anchor – 6 strands visible with deep pits, 18” to 28” of penetration, bright copper, orange corrosion on tendons, active corrosion on side of trumpet
South Anchor – 2 ½” void then solid grout, no video

Photographs of removed tendon:


4.2.4 Tendon 9-1

Span Type: Expansion Joint Span
Date Stressed: 6/16/92
Date Grouted: 6/24/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

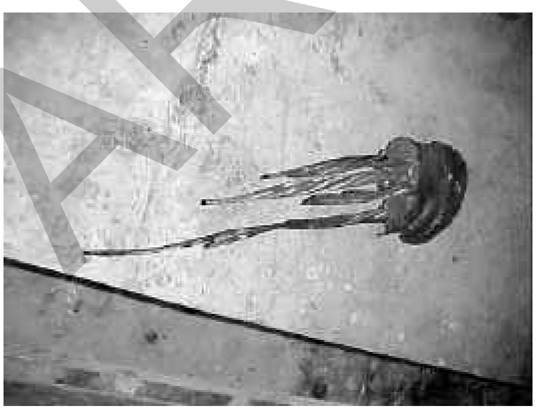
Borescope Photographs:

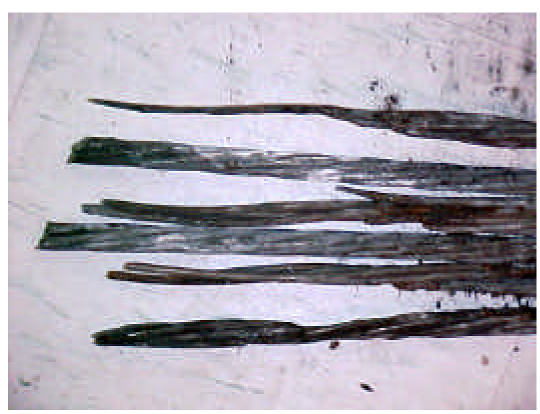


Borescope Field Notes:

North Anchor – 3 to 4 strands visible, black and gray heavy corrosion on bottom of strands, broken grout, red and black (active) corrosion.
South Anchor - Tendon under replacement at time of inspection, no video or notes.

Photographs of removed tendon:

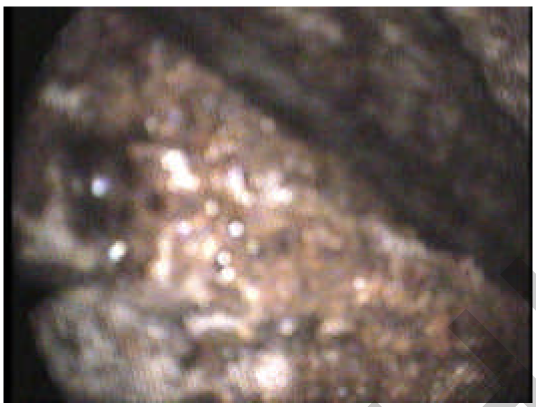


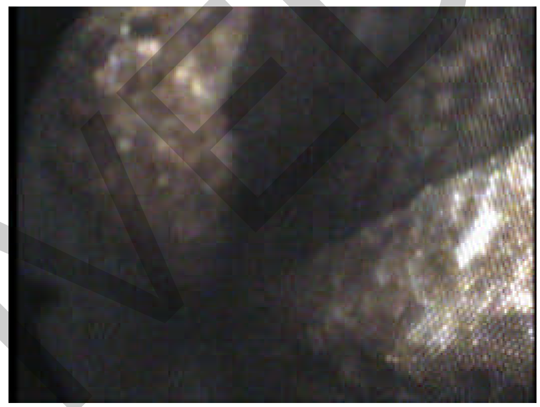


4.2.5 Tendon 58-5

Span Type: Expansion Joint Span
Date Stressed: 9/19/92
Date Grouted: 9/25/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

Borescope Photographs:





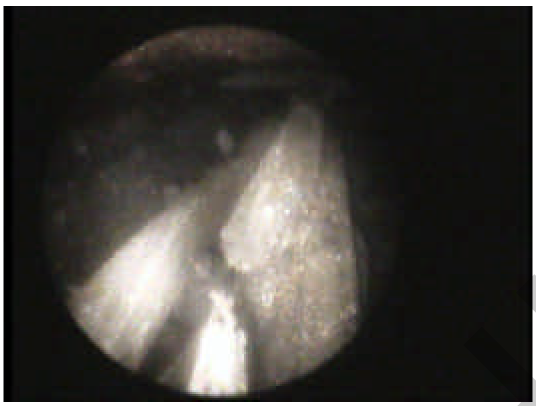
Borescope Field Notes:

North Anchor – no corrosion, white grout
South Anchor – no grout present, 8 to 10 strands visible, severe corrosion, active corrosion cells, wires on strands could not be distinguished due to corrosion for approximately 4” to 6”

4.2.6 Tendon 63-6

Span Type: Expansion Joint Span
Date Stressed: 9/25/92
Date Grouted: 10/8/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

Borescope Photographs:



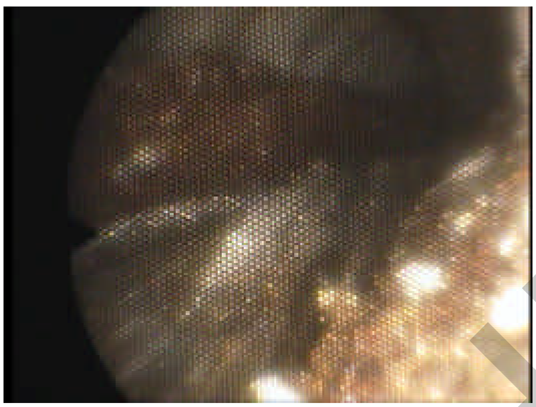
Borescope Field Notes:

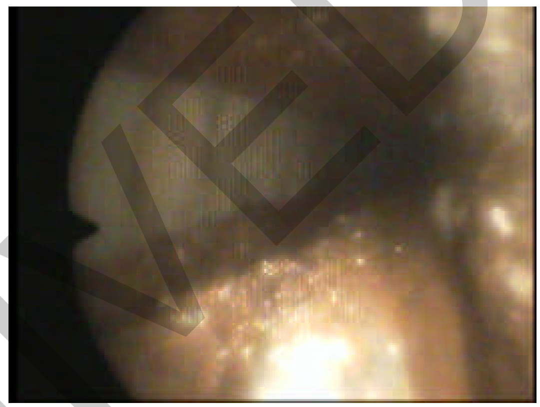
North Anchor – 5' void, 5 strands visible, trumpet has advanced corrosion with pitting, several strands with advanced corrosion and what appears to be pitting.
South Anchor – 3 strands visible with light orange spotty corrosion, moderate corrosion on trumpet, white grout, and 2' penetration.

4.2.7 Tendon 69-3

Span Type: Expansion Joint Span
Date Stressed: 10/19/92
Date Grouted: 10/21/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

Borescope Photographs:





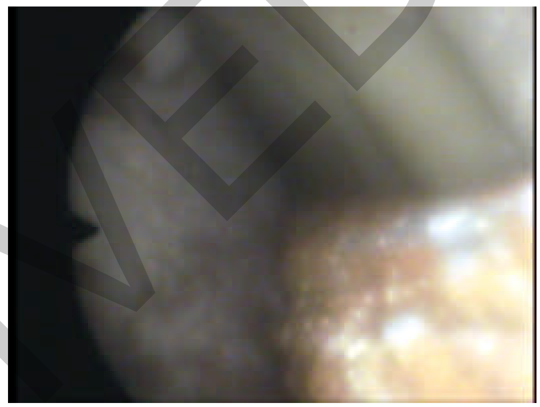
Borescope Field Notes:

North Anchor – 3' void, 5 to 7 strands visible, appears to be necking
South Anchor – small void, black corrosion on trumpet, 8" void

4.2.8 Tendon 69-2

Span Type: Expansion Joint Span
Date Stressed: 10/19/92
Date Grouted: 10/21/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

Borescope Photographs:



Borescope Field Notes:

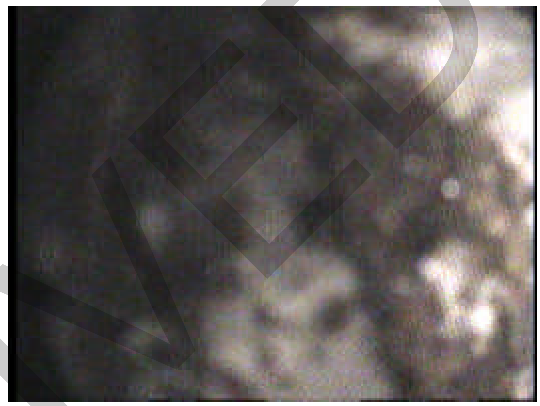
North Anchor – 18" void, 5 strands visible, extremely heavy corrosion, active corrosion cells on strands
South Anchor – good white grout, 6" void

Photographs of removed tendon:

4.2.9 Tendon 64-1

Span Type: Expansion Joint Span
Date Stressed: 9/30/92
Date Grouted: 10/8/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

Borescope Photographs:



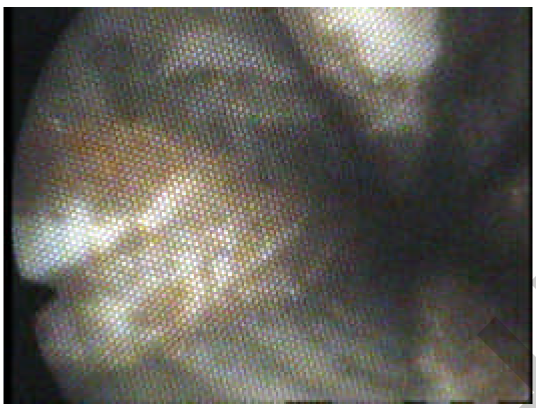
Borescope Field Notes:

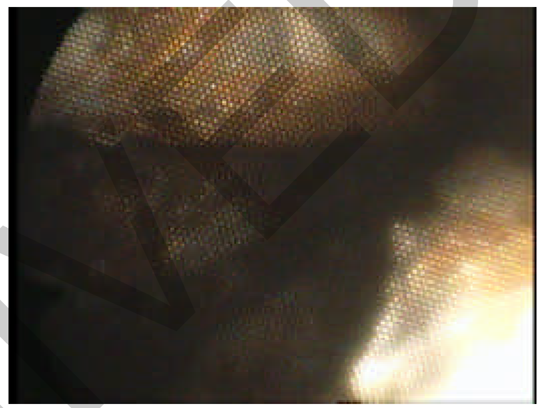
North Anchor – moderate corrosion on trumpet, no strands visible, white grout, 1'-6" void
South Anchor – 3' void plus, 5 strands visible, wires on strands cannot be distinguished, severe corrosion present, active corrosion cells. Face of diaphragm at pier 64 has three diagonal cracks adjacent to all of the ducts, effervescence present at the top of deck underside adjacent to duct 64-1.

4.2.10 Tendon 58-6

Span Type: Expansion Joint Span
Date Stressed: 9/19/92
Date Grouted: 9/25/92
Date Expansion Joint Placed: Specific date of expansion joint placement not determined from construction records reviewed. Records indicate that expansion joint placement began at the South Abutment on 10/8/92, with work progressing from south to the north.

Borescope Photographs:





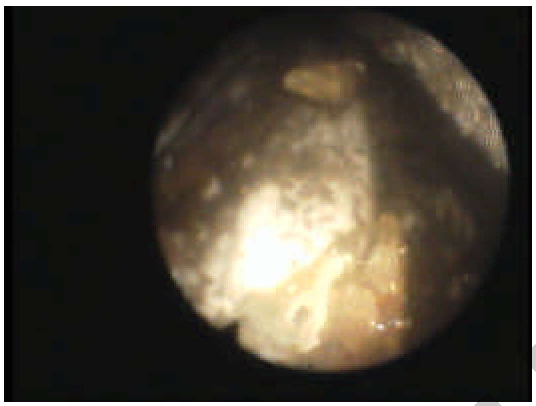
Borescope Field Notes:

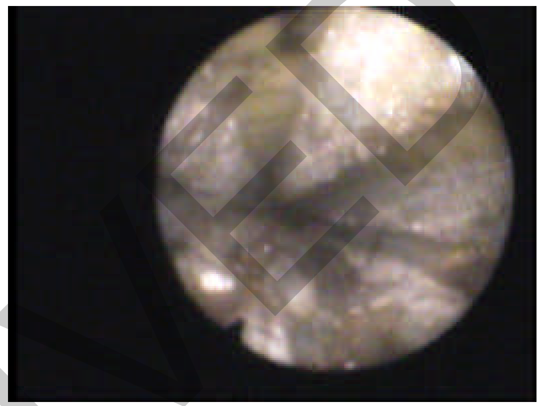
North Anchor – no corrosion, white grout
South Anchor – 3’-6” void, 8 to 9 strands visible, severe corrosion, active corrosion cells, wires on strands could not be distinguished due to corrosion for approximately 12”

4.2.11 Tendon 48-5

Span Type: Interior Span
Date Stressed: 9/4/92
Date Grouted: 9/10/92
Date Next Span Erected: Span 49 was stressed on 9/8/92

Borescope Photographs:





Borescope Field Notes:

North Anchor – 12” void, white grout, light red corrosion on trumpet
South Anchor – 5 to 6 visible strands, 1 strand has a broken wire, moderate to heavy corrosion on all strands with pitting and blistering, moderate to heavy corrosion on trumpet, white grout

4.2.12 Removal and Replacement of Span-By-Span, External Post-Tensioning Tendons

The following are the steps taken by the Contractor to remove an external tendon from the Mid-Bay Bridge:

1. Remove the PE pipe from the entire length of the tendon.
2. Locally remove grout and install 4-inch diameter heavy-duty U -bolt clamps every 4 ft. on the tendon to control the possible strand 'whiplash' as each strand is cut.
3. Remove as much grout as practical throughout the entire length of the tendon.
4. Remove as much grout as possible from the steel ducts at the deviation diaphragms using a high-pressure hydro-blaster (to decrease bond at the deviation diaphragms).
5. Strand cutting will be performed with an electric powered cut-off saw using metal abrasive blades. Torch cutting will not be allowed.
6. Cut one strand of the tendon at the down station side of the down station deviation diaphragm. (Leaving enough strand length so that a mono-strand jack can be used to grip the strands and remove them later)
7. Cut one strand at location up station side of the down station deviation diaphragm.
8. Repeat Steps 6 and 7 at the up-station deviation diaphragm.
9. Repeat Steps 6, 7, and 8 never allowing more than one strand cut out of balance at any deviation diaphragm.
10. Check that cut strands are shortening by the appropriate amount to relieve their force. If not, loosen U-bolt clamps to allow cut strands to slide along their length.
11. When all strands are cut, use the remaining tails to pull out the strand at the deviation saddles.
12. Use hydro-blaster to remove grout in the pier segment diaphragms.
13. Remove anchor plate using air assisted arc cutter to control amount of heat required. Acetylene torches generally pop more around cementitious grout and destroy tips on the torch. Ventilation was critical for worker safety.
14. Use tails of strands to remove the tendon from the pier segment diaphragms.
15. Place new polyethylene duct.
16. Push strands in place, starting from the bottom of the duct. Use a locating plate to guide the strands in the duct to prevent twisting of the tendon bundle in the duct.
17. Pull initial load on all strands using monostrand jack, starting at the top of the tendon.
18. Apply final stressing of strands working from the strands in the top of the duct to the strands at the bottom. (Use of a multi-strand jack is permitted for Steps 17 and 18 if access and clearances are sufficient).
20. Cut tails and cap vents and ports in the tendon duct within 4 hours after stressing*.
21. Grout tendon within 7 days*.
22. Leave grout cap on as protection for a minimum of 72 hours after grouting*.
23. Remove grout cap and inspect for voids*.
24. Cast pour-back within a minimum of 54 hours following inspection for voids*.
25. Apply mastic protective covering within 4 hours of removing forms of the pour-back*.

* In accordance with the new FDOT post-tensioning specification.

The total cost to replace the 11 tendons in the Mid-Bay Bridge was $999,680.

Figure 4.1 – Details of the Removal of External Tendons (Clockwise from upper left: clamping strands, cutting cable, hydro-blasting at pier segment diaphragm, and hydro-blasting operator)

4.3 Repair of Tendon Anchorages

All grout injection ports were sealed after borescope inspections to limit the entrance of additional moisture into the voided anchors. The following is a description of the approved methods and materials developed by the FDOT Central Structures Office for the repair of the post-tensioning system of the Mid-Bay Bridge.

Various methods for cleaning the voids prior to re-injection were investigated. Consideration was given to cleaning by flushing the voids with either water alone or water with a concentration of lime. Protecting strands by placing corrosion inhibitors was also studied. The use of corrosion inhibitors would require that some water be introduced during application or that water be used to clean the strands prior to grouting. Based on the apparent condition of the existing grout and possible wicking action of the strands, the FDOT decided that adding any water to the voids could do more harm than good. As a result, the voids will only be prepared by removing debris using compressed air.

After a detailed inspection, the following list of needed repairs was established (for a final listing of quantities actually used in the field, see Chapter 6):

A. Replace all pour-backs located at expansion joint piers - 89 required

B. Grout anchorage voids with strands visible - 274 required
C. Grout anchorage voids without strand visible - 316 required
D. Replace pour-back at interior piers - 307 required
E. Coat undamaged pour-backs with coal tar epoxy - 408 required

4.3.1 Replacement of Pour-Backs at Expansion Joint Piers

Remove all coal tar epoxy from expansion joint pier segments by mechanical cleaning. Remove grout cap material and any scaling corrosion products to bare metal by mechanical cleaning. Immediately after cleaning, form the new pour-back and cast full using a flow and fill epoxy compound. Coat the pour back and adjoining concrete surface with coal tar epoxy.

4.3.2 Vacuum Grouting of Anchor Voids

Clean the void by blowing compressed air through the grout port using a wand. Continue blowing air into the void until debris and dust stop exiting the grout port. After cleaning, prepare void for vacuum injection by sealing all air leaks. Vacuum inject grout the void (See Section 2.7 for injection procedures).

4.3.3 Replacement of Pour-Back at Interior Piers

Remove grout cap material and any scaling corrosion products to bare metal by mechanical cleaning. Install non-metallic grout cap that mounts to the exposed face of the multi-plane anchor. This grout cap covers the strands and wedge plate allowing for complete encapsulation of the anchor hardware. Using a tube completely fill the grout cap with cementitious grout. Apply two coats of coal tar epoxy to the grout cap and the adjoining concrete surface.

4.3.4 Sealing of Existing Pour-Backs

Sound pour-back with hammer for solid or hollow response. Visually inspect anchorage for signs of corrosion. If a hollow response or corrosion is observed, remove the pour back and replace in accordance with the procedures of Section 4.3.3. If a solid response and no corrosion are observed, apply two coats of coal tar epoxy to the pour-back and the adjoining concrete surface.

4.3.5 Approved Materials

All materials shall be used in strict accordance with the manufacturers instructions.

Cement Grout:	Master Flow 816 Cable Grout
Coat Tar Epoxy:	Bitumastic 300M
Grout Cap:	DSI Grout Cap 68197210 and "0" ring gasket. Use this grout cap at all locations except at expansion joints.
Epoxy Grout:	Ceilcote 648 CP Plus

4.4 Duct Wrapping

The loss of post-tensioning tendons to corrosion elevates the cracking of the duct from a maintenance issue to one of fundamental importance. The polyethylene duct serves as the outer defense for the corrosion protection of the prestressing strands. Several locations of localized corrosion in the Mid-Bay Bridge were found where the ducts were punctured, in spite of being filled with grout. One of the failed tendons, Tendon 28-6 failed in the free length of tendon where only the grout and polyethylene duct are providing protection.

As a result of the extensive cracking of the ducts, a significant program of wrapping the ducts of the bridge is planned. The total length of duct to be wrapped is approximately 104,000 linear feet. To date, approximately 17,000 linear feet have been wrapped, leaving approximately 87,000 linear feet to be wrapped. Of the 17,000 linear feet of ducts already wrapped, 12,000 linear feet were wrapped under a previous construction contract (FIN 220223-1-52-01).

The polyethylene ducts were wrapped with Wrapid Sleeve by CANUSA. The wraparound sleeves consist of a cross-linked polyolefin backing coated with a protective heat sensitive adhesive. Individual sleeve sheets are wrapped around the duct and then heat is applied to activate the adhesive. Figure 4.2 shows the heating of a wraparound sleeve. Figure 4.3 shows a completed wrapping of an external tendon. Note the direction of overlapping of the sleeve sheets to prevent water that may collect on the wrapping at joints.

Figure 4.2 – Heat activating the adhesive in the polyethylene duct wrapping.

Figure 4.2 – Heat Activated Adhesive Duct Wrapping for External Post-tensioning Tendons

Chapter 5 – Structural Analyses

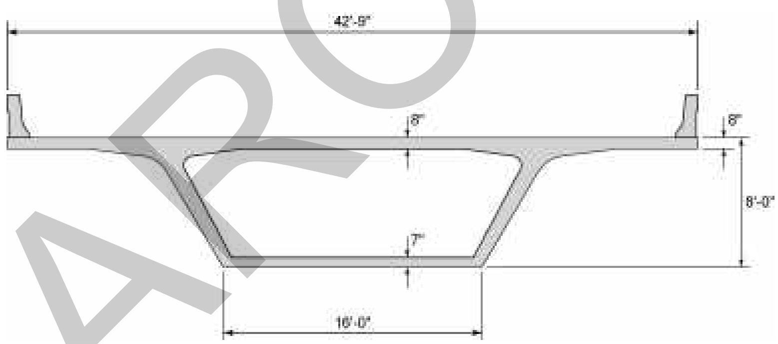
5.1 Introduction

Structural analyses were performed for a typical 6-span unit of the Mid-Bay Bridge. These analyses were undertaken to better understand the behavior of the bridge including the effects of the loss of prestress force as a result of corrosion. This work was accomplished using the Bridge Designer II (BDII) computer program. The BDII program models the bridge components and construction staging consistent with the actual construction of the Mid-Bay Bridge. BDII also evaluates traffic effects on the bridge that represent the design live loadings and legal rating vehicles used by the Florida Department of Transportation (FDOT). The results presented in this Chapter are Rating Factors for the design and legal trucks, for various configurations of prestressing in a typical 6-span unit.

5.2 Analysis Parameters

The typical 6-span unit is made of four interior typical spans and two expansion joint spans at either end of the unit. The span length of the typical spans is 136' and the span length of the expansion joint spans (to the centerline of the expansion joint bearings) is 133'-6". The distribution of the segments in the typical and expansion joint spans is shown in Chapter 1, Figure 1.4.

The cross section used in the modeling of the typical 6-span unit is the single-cell box girder shown in Figure 1.2. This cross section is presented again in Figure 5.1 along with the cross section properties used in the structural analyses.



Area = 58.13 ft^2
c_{top} = 2.4734 ft
c_{bottom} = 5.5266 ft
Inertia = 487.84 ft^4
Efficiency = 0.6139

Figure 5.1 – Mid-Bay Bridge Cross-Section Properties

The analysis made by the BDII program is a two-dimensional, time-dependent frame analysis of a bridge model. Nodes, which have three degrees of freedom for displacement (vertical, horizontal and in-plane rotation), are defined at specific locations to model the bridge geometry. The nodes are related to each other in the model by the definition of frame elements that have desired member characteristics. Nodes for the typical 6-span model of the Mid-Bay Bridge are located at joints between the precast elements, closure joints, and at support locations. The frame element characteristics are those of the typical cross section presented in Figure 5.1.

Post-tensioning tendons are defined as per the details presented in the design drawings, with consideration given to the location of strands within the ducts.

An analysis using the BDII program includes the time-dependent characteristics of the concrete and prestressing steel used to build concrete segmental bridges. Using an iterative approach, the effects of concrete creep, concrete shrinkage and prestressing steel relaxation on the state of stress in the bridge are evaluated. The variations of these characteristics used for these analyses are those presented in the FIP-CEB Model Code used in Europe. This is the same approach taken in the original design.

Each time-dependent analysis establishes a timeframe relative to the casting and erection of the portion of bridge being analyzed. For this study, the following timeframe was established based on project documentation for the average ages of all of the segments at the time of erection.

Casting Date of All Segments of the unit – Day 0
Erect Span 1 – Day 50
Erect Span 2 – Day 52
Erect Span 3 – Day 54
Erect Span 4 – Day 56
Erect Span 5 – Day 58
Erect Span 6 – Day 60
Place Barrier Railing – Day 74

External loadings for the analyses were taken from the General Notes of the design plans. Barrier Railing loads and weights of internal diaphragms were computed from the plan concrete dimensions. The thermal gradient used in the original design was an 18°F linear gradient (top slab warmer than bottom slab). Though current code requirements are slightly different, these analyses used the same 18°F linear gradient for comparative purposes.

5.3 Code Changes and Load Rating

Developing the load ratings for a concrete segmental bridge is an involved process that begins with the time-dependent analysis and concludes with the verification of each section of the bridge with regard to the effects of the different rating vehicles. This effort can be further complicated by the fact that design code requirements governing segmental bridges have changed since first introduced.

The Mid-Bay Bridge is one of several bridges in Florida whose ratings today are affected by updates to governing codes. The "*Guide Specification for Design and Construction of Segmental Concrete Bridges"* (Guide Specifications) was developed in 1988 as a NCHRP project and subsequently adopted as a guide specification by the Highway Subcommittee on Bridges and Structures of AASHTO in 1989. The Mid-Bay Bridge was under design in 1989 and the first span of the bridge was stressed on May 16, 1992. In 1999 AASHTO approved the 2nd Edition to the guide specifications, incorporating interim revisions to the 1st Edition and input from a committee that consisted of state and federal highway officials, consultants, contractors, suppliers, and academicians. FDOT practice is to rate bridges in accordance with current applicable codes and FDOT publication "*Bridge Load Rating, Permitting and Posting Manual."*

Significant changes in the Guide Specifications with regard to the Mid-Bay Bridge are:

Service Load Flexure:	The 1st Edition permitted reduction of variable load effects (interpreted as the live loads and gradients in the original design) by the overstress factors in the AASHTO Code and allowed zero tension for bridges with dry (non-epoxied) joints.
	The 2nd Edition does not permit reduction of variable load effects and requires a 100 psi residual compression in bridges with Type B (non-epoxied) joints.
Ultimate Flexural Strength:	The 1st Edition included gradient effects in ultimate load combinations and allowed an increase of 15 ksi in the stress in prestressing steel at ultimate for unbonded tendons.
	The 2nd Edition assigns a load factor of zero to gradient effects in ultimate load combinations and allows an increase in prestressing steel stress as a function of free length of tendon for deviated external tendons. The AASHTO Code supplies an upset limit to this stress increase equal to the yield stress of the prestressing (0.9 of the ultimate strength for low-relaxation steel).
Shear:	The 1st Edition used a capacity reduction factor for shear of 0.9.
	The 2nd Edition reduced the capacity reduction factor for shear to 0.85.

5.4 Load Rating and Parametric Study Results

Load ratings and parametric studies were performed for a typical 6-span unit of the Mid-Bay Bridge. The load ratings were conducted in accordance with FDOT publication "*Bridge Load Rating, Permitting and Posting Manual.*" The typical 6-span unit with original post-tensioning was rated with and without the effects of the future wearing surface as per the project plans. Other parameters of the load ratings were:

- Inventory ratings were developed for the HS-20 Truck only.
- Operating ratings were developed for the HS-20 Truck and the seven legal trucks defined in the FDOT load rating publication.
- Flexural load ratings at inventory and operating level were developed considering zero tension at the joints between precast segments.
- Shear load ratings were performed at load factor level considering the appropriate load magnification for inventory and operating ratings.
- A capacity reduction factor (ø) equal to 0.85 was used in the shear load ratings.

The results of the load ratings for shear and flexure, with and without future wearing surface are given in Table 5.1 and Table 5.2, respectively. These results are expressed as Rating Factors that give a relative measure of the number of loadings that the bridge can support. A Rating Factor equal to 1.0 would indicate that the bridge could support the vehicle in question placed in each of the three design lanes simultaneously, with the appropriate 0.9 lane reduction factor. The bridge ratings are also given in terms of the number of individual design lanes that the typical unit can support. These values are given in parentheses in Tables 5.1 and 5.2. The number of single design lanes (the values in parentheses) is found by multiplying the Rating Factor by 2.7 (3 lanes x 0.9 reduction = 2.7).

		Ratings without Future Wearing Surface					
		Flexure		Shear		Governing	
Inventory Ratings		1.27	(3.43)	1.32	(3.56)	1.27	(3.43)
Operating Ratings	HS20 Truck	1.27	(3.43)	2.37	(6.40)	1.27	(3.43)
	SU2 Truck	2.34	(6.32)	4.48	(12.10)	2.34	(6.32)
	SU3 Truck	1.27	(3.43)	2.32	(6.26)	1.27	(3.43)
	SU4 Truck	1.18	(3.19)	2.23	(6.02)	1.18	(3.19)
	C3 Truck	1.78	(4.81)	2.95	(7.97)	1.78	(4.81)
	C4 Truck	1.35	(3.65)	2.46	(6.64)	1.35	(3.65)
	C5 Truck	1.44	(3.89)	2.36	(6.37)	1.44	(3.89)
	ST5 Truck	1.45	(3.92)	2.72	(7.34)	1.45	(3.92)

Table 5.1 – Load Rating for 6-span Unit of the Mid-Bay Bridge with no wearing surface.

		Ratings with Future Wearing Surface					
		Flexure		Shear		Governing	
Inventory Ratings		1.20	(3.24)	1.20	(3.24)	1.20	(3.24)
Operating Ratings	HS20 Truck	1.25	(3.38)	2.16	(5.83)	1.25	(3.38)
	SU2 Truck	2.38	(6.43)	4.08	(11.02)	2.38	(6.43)
	SU3 Truck	1.24	(3.35)	2.11	(5.70)	1.24	(3.35)
	SU4 Truck	1.17	(3.16)	2.03	(5.48)	1.17	(3.16)
	C3 Truck	1.61	(4.35)	2.68	(7.24)	1.61	(4.35)
	C4 Truck	1.25	(3.38)	2.24	(6.05)	1.25	(3.38)
	C5 Truck	1.26	(3.40)	2.15	(5.81)	1.26	(3.40)
	ST5 Truck	1.24	(3.35)	2.47	(6.67)	1.24	(3.35)

Table 5.2 – Load Rating for 6-span Unit of the Mid-Bay Bridge with wearing surface.

The Rating Factors presented in Tables 5.1 and 5.2 indicate that the bridge, in pristine condition, would be able to carry the design live load (HS-20 truck) as well as the weight of more than one of each of the seven legal trucks in the appropriate number of design lanes.

Parametric studies were made to model the effects of the loss of post-tensioning tendons due to corrosion. Unit 10, which contains Spans 52 through 57, was considered. Tendon 57-1 had failed completely due to corrosion and Tendon 57-2 was subsequently replaced after inspection revealed extensive deterioration. Span 57 is represented as Span 6 in the computer models developed. The following combinations of prestressing configurations were considered:

- All spans constructed with all post-tensioning in place.
- All spans constructed, all tendons stressed, then Tendon 6-1 removed.
- All spans constructed, all tendons stressed, then Tendons 6-1 and 6-2 removed.
- All spans constructed, all tendons stressed and Tendons 6-1 and 6-6 removed. This case was considered for shear only, in order to investigate the loss of those tendons most beneficial to resisting shear at the expansion joint end of the expansion joint spans.

Table 5.3 shows the impact of the reduction of post-tensioning in Span 6 on flexural load rating, expressed in terms of Rating Factors. As in Tables 5.1 and 5.2, the numbers in parentheses represent the number of individual design lanes that can be supported by the bridge according to the FDOT rating guidelines.

		Flexural Parametric Study - No Wearing Surface					
		Full PT		T1 Removed		T1 & T2 Removed	
Inventory Ratings		1.27	(3.43)	0.71	(1.92)	-0.03	-(0.08)
Operating Ratings	HS20 Truck	1.27	(3.43)	0.71	(1.92)	-0.03	-(0.08)
	SU2 Truck	2.34	(6.32)	1.42	(3.83)	-0.06	-(0.16)
	SU3 Truck	1.27	(3.43)	0.74	(2.00)	-0.03	-(0.08)
	SU4 Truck	1.18	(3.19)	0.69	(1.86)	-0.03	-(0.08)
	C3 Truck	1.78	(4.81)	0.96	(2.59)	-0.04	-(0.11)
	C4 Truck	1.35	(3.65)	0.74	(2.00)	-0.03	-(0.08)
	C5 Truck	1.44	(3.89)	0.75	(2.03)	-0.03	-(0.08)
	ST5 Truck	1.45	(3.92)	0.74	(2.00)	-0.03	-(0.08)

Table 5.3 – Effect of Post-Tensioning Loss on Rating Factors not including wearing surface, including effects of thermal gradient.

The results of the parametric study shown in Table 5.3 indicates that Span 6 can support only 71% of the design lanes at inventory level, with no tension at the joints when Tendon 6-1 is removed. This represents 1.92 individual design lanes of the AASHTO HS20 Truck (0.71 x 3 x 0.9). Or in terms of two lanes, the Mid-Bay Bridge can support two lanes of HS19 trucks when one tendon is removed from the expansion joint spans.

Table 5.3 shows negative values when both Tendon 6-1 and Tendon 6-22 are removed from Span 6. These negative results indicate that there is no live load capacity in the bridge <u>with respect to joint openings</u> with two tendons removed in an expansion joint span. The results indicate that the joints would open in this span under the influence of thermal gradient only.

Table 5.4 is similar to Table 5.3 in that it presents the effects of the loss of post-tensioning on the flexural capacity of Span 6 in the typical 6-span unit of the Mid-Bay Bridge. The values in this table do not include the effects of thermal gradient.

		Flexural Parametric Study - No Wearing Surface					
		Full PT		T1 Removed		T1 & T2 Removed	
Inventory Ratings		1.56	(4.21)	0.87	(2.35)	0.13	(0.35)
Operating Ratings	HS20 Truck	1.56	(4.21)	0.87	(2.35)	0.13	(0.35)
	SU2 Truck	3.10	(8.37)	1.73	(4.67)	0.26	(0.70)
	SU3 Truck	1.61	(4.35)	0.90	(2.43)	0.14	(0.38)
	SU4 Truck	1.52	(4.10)	0.85	(2.30)	0.13	(0.35)
	C3 Truck	2.09	(5.64)	1.17	(3.16)	0.17	(0.46)
	C4 Truck	1.62	(4.37)	0.91	(2.46)	0.14	(0.38)
	C5 Truck	1.63	(4.40)	0.91	(2.46)	0.14	(0.38)
	ST5 Truck	1.61	(4.35)	0.90	(2.43)	0.13	(0.35)

Table 5.4 – Effect of Post-Tensioning Loss on Rating Factors not including effects of thermal gradient.

The effect of the loss of post-tensioning tendons in Span 6 on shear capacity is presented in Table 5.5. Although capacity reduces with the loss of post-tensioning, the impact on the ability of the Mid-Bay Bridge to carry shear is not as pronounced as for resistance to flexure. This is primarily the result of good shear characteristics of the concrete box girder and the ability to rate shear at ultimate load levels. It is important to note that actual behavior of the end of the expansion joint span will be somewhat different from these results as the 3-dimensional effects of torsion, out of plane bending, distribution of prestressing forces, and shear lag are not included.

		Shear Parametric Study - No Wearing Surface							
		Full PT		T1 Removed		T1 & T2 Removed		T1 & T6 Removed	
Inventory Ratings		1.32	(3.56)	1.17	(3.16)	1.00	(2.70)	0.92	(2.48)
Operating Ratings	HS20 Truck	2.37	(6.40)	2.01	(5.43)	1.69	(4.56)	1.52	(4.10)
	SU2 Truck	4.48	(12.10)	3.81	(10.29)	3.19	(8.61)	2.87	(7.75)
	SU3 Truck	2.32	(6.26)	1.97	(5.32)	1.65	(4.46)	1.49	(4.02)
	SU4 Truck	2.23	(6.02)	1.89	(5.10)	1.59	(4.29)	1.43	(3.86)
	C3 Truck	2.95	(7.97)	2.50	(6.75)	2.09	(5.64)	1.89	(5.10)
	C4 Truck	2.46	(6.64)	2.09	(5.64)	1.75	(4.73)	1.58	(4.27)
	C5 Truck	2.36	(6.37)	2.00	(5.40)	1.68	(4.54)	1.51	(4.08)
	ST5 Truck	2.72	(7.34)	2.30	(6.21)	1.93	(5.21)	1.74	(4.70)

Table 5.5 – Effect of Post-Tensioning Loss on Rating Factors

5.5 Structural Analyses and FDOT Actions

Immediately following the discovery of the failed post-tensioning tendons in Span 28 and 57, the Florida Department of Transportation took important steps to assure safe operation of the Mid-

Bay Bridge. Two important actions taken were two closures of the bridge to all traffic and two other closures of the bridge to truck traffic (see Section 1.3).

Initial calculations, developed according to the 1st Edition of the Segmental Guide Specifications (see Section 5.3), were prepared to justify two lanes of traffic on the bridge with one post-tensioning tendon removed in a span. Though not in agreement with the use of the 1st Edition for the evaluation of load carrying capacity, the FDOT did realize these calculations indicated there was no live load capacity, with respect to joint openings, when two tendons were removed from a span.

The immediate response of closing the bridge on August 28th and 29th to all traffic allowed the FDOT time to perform vibration testing on the remaining five tendons in each of Spans 28 and 57, without risking the failure of a second tendon with traffic on the bridge. Based on the results of this vibration testing, the bridge was re-opened to two-axle vehicles. From August 29th to September 11th the FDOT developed procedures to remove a partially stressed post-tensioning tendon. On September 11th it was decided that the vibration testing would not be solely relied upon to establish confidence in the other five tendons in Span 28. As a result, selected borescope testing was performed in this span. These inspections confirmed that two lanes of traffic with 2-axle vehicles only could use the bridge during replacement of Tendon 28-6.

From September 11th to September 26th several activities were underway at the bridge site. Construction crews were replacing Tendons 28-6 and 57-1, grout cap damage was being inventoried, and borescope inspections of Spans 1 through Span 9 and other random locations were performed. The severity of the corrosion found in the borescope inspections lead the FDOT to recommend to the Mid-Bay Bridge Authority to close the bridge completely so thorough inspections could be performed. The Mid-Bay Bridge Authority closed the bridge to all traffic from September 27th to October 11th.

Inspection crews were assembled from around the state, and work began to inspect with a borescope every anchor along with vibration testing of every tendon while the bridge was closed from September 27th to October 11th. Construction crews continued tendon removal and installation activities during this bridge closure as tendons were identified for replacement. On October 11th enough information had been gathered and enough repairs had taken place to again have confidence in the bridge's ability to carrying two lanes of two-axle vehicles.

The load ratings and parametric studies presented in Section 5.4 were performed subsequent to the FDOT response to the tendon failures, and were not available to assist the FDOT in determining the capacity of the bridge during tendon replacement. These studies do, however, confirm FDOT actions to close the bridge when one tendon in a span is failed and the condition of the other tendons in that span is suspect. These actions were further affirmed when three spans, Spans 57, 58, and 69, were each found to have second tendons that required replacement.

The analytical studies presented in this report also support the FDOT position of allowing only two axle vehicles on the bridge during later tendon replacement. This is seen in the results presented in Table 5.3 where only the SU2 vehicle rates higher than 1.0 when one tendon is removed.

The analytical studies of the typical 6-span unit of the Mid-Bay Bridge is part of a larger effort to rate the longitudinal flexural and shear behavior of all continuous units of the bridge. Superstructure calculations are provided in Appendix G of this report.

Chapter 6 – Summary of Post-Tensioning Repairs

This Chapter summarizes the actual quantities of the various repairs made to the post-tensioning system of the Mid-Bay Bridge. Details of the various quantities are presented in Appendix H of this report.

6.1 Tendon Replacements

Eleven tendons each comprised of 19, 0.6” diameter prestressing strands were replaced. The tendon numbers and the dates stressed are:

Tendon	Date Stressed
Tendon 57-1	9/19/00
Tendon 28-6	9/19/00
Tendon 57-2	10/11/00
Tendon 9-1	10/12/00
Tendon 58-5	10/24/00
Tendon 63-6	10/24/00
Tendon 69-3	10/24/00
Tendon 69-2	11/07/00
Tendon 64-1	11/07/00
Tendon 58-6	11/07/00
Tendon 48-5	11/15/00

6.2 Anchor Cap Replacements

Anchors of interior piers requiring protection were covered by a Dywidag Systems International plastic grout cap that was filled with Master Builders 1205 grout. The total number of anchors protected was 724.

6.3 Expansion Joint Anchor Protection

Tendon anchorages at expansion joint piers were protected by an encapsulating pour-back of Ceilcote 648 CP Plus epoxy grout. The total number of anchors protected was 300.

6.4 Tendon Duct Wrapping

The majority of the polyethylene duct splitting and punctures will be repaired under the recently bid repair contract (bid August 31, 2001). Some wrapping was performed, however, during emergency repairs and vacuum injection of anchors. The length of duct wrapped during the emergency repairs was 3,640 linear feet. The length of duct wrapped to seal tendons for vacuum injection was 945 linear feet.

6.5 Vacuum Injection

A specialty subcontractor assisted the prime contractor in vacuum injecting the anchors. A total of 679 anchors were injected with a total volume of 2052.4 liters of Master Builders 816 grout.

Of the total 679 anchors that were injected in the entire bridge, 650 of these were in the typical 136' spans built by the span-by-span method of construction. The total number of anchorages in these typical spans is 1,656. This volume of grout injected in the typical spans was 1991.9 liters. An analysis of the anchors injected in these spans is provided in the following sections.

6.5.1 Interior Pier Anchorages

There are a total of 1368 interior pier anchors in the 138 spans built by the span-by-span method of construction. Of these anchors, 571 were vacuum injected. This represents 42 percent of the interior pier anchorages. The total volume of grout injected in the interior pier anchors was 1865.7 liters. The distribution of grout volume injected in the interior pier anchors, in liters, relative to the tendon type was:

	T1	T2	T3	T4	T5	T6
Volume	384.2	291.1	241.8	326.5	266.8	355.3
Percent	20.6%	15.6%	13.0%	17.5%	14.3%	19.0%

The distribution of the number of anchors grouted and the average volume of grout placed in each anchorage, in liters, relative to the tendon type was:

	T1	T2	T3	T4	T5	T6
Number	101	111	92	82	92	93
Volume	3.8	2.6	2.6	4.0	2.9	3.8

The maximum amount of grout injected at an interior anchor was 19.5 liters at Tendon 68-1 north. A review of the borescope log of this anchor indicated the following conditions:

> *"Voids in grout, 1 strand exposed (no corrosion), white grout; gravelly grout, black corrosion on trumpet, 3' penetration"*

6.5.2 Expansion Joint Pier Anchorages

There are a total of 288 expansion joint pier anchors in the 138 spans built by the span-by-span method of construction. Of these anchors, 79 were vacuum injected. This represents 27 percent of the expansion joint pier anchors. The total volume of grout injected in these anchors was 126.2 liters. The distribution of grout volume injected in the expansion joint pier anchors in liters relative to the tendon type was:

	T1	T2	T3	T4	T5	T6
Volume	24.0	17.0	8.0	4.2	28.0	45.0
Percent	19.0%	13.5%	6.3%	3.3%	22.2%	35.7%

The distribution of the number of anchors grouted and the average volume of grout placed in each anchorage in liters was:

	T1	T2	T3	T4	T5	T6
Number	18	10	11	5	13	22
Volume	1.3	1.7	0.7	0.8	2.2	2.0

The maximum amount of grout injected at an expansion joint anchor was 8.0 liters at Tendon 22-6 south. A review of the borescope log of this anchor indicated the following conditions:

> *"Grout has a void approximately 4'+, white grout, 6 to 8 stands visible with (red) light corrosion, moderate (red) corrosion on trumpet, spotted corrosion on bottom of trumpet"*

6.5.3 Other Comparisons

- The number of interior pier anchors injected and the corresponding volume of grout injected on the 81 spans south of the three span main unit were compared to the 57 spans north of the main unit. The average number of interior pier anchors injected per span was 5.0 in the south spans and 2.9 in the north spans. The average volume of grout injected in the interior pier anchors was 19.7 liters in the south spans and 4.7 liters in the north spans. The total number of anchors per span is 12.

- A review of Section 6.4.1 and Section 6.4.2 indicates that voids were typically larger at interior piers, though more significant corrosion and the majority of the tendon replacements were at expansion joint piers. It is important to remember that the numbers and volumes in Section 6.4.2 do not include those of the tendons that were replaced.

- Even without the void sizes of the removed tendons, the location and distribution of a void appears to be more important than the absolute void size. Smaller concentrated voids at the inclined anchorages of the expansion joints expose more strands to concentrated volumes of bleed and recharged water. Larger voids at interior piers with horizontal tendon profiles near the anchorages are thinly distributed throughout the pier segment diaphragms. Fewer strands are exposed and the opportunity for recharge is small because the deck is continuous over these anchors.

Chapter 7 – Costs Associated With Bridge Repairs

This Chapter presents the cost information related to the repairs of the post-tensioning system of the Mid-Bay Bridge. All construction activities have been concluded with the exception of a tendon duct wrapping contract that was let on August 31, 2001.

Figure 7.1 presents a summary of the costs as of September 4, 2001. The cost of the construction repairs, not including the tendon duct wrapping contract let on August 31, 2001, is $2,587,861.45. The apparent low bid for the wrapping contract is $1,482,588.18. Engineering costs associated with the project are $657,340.47. This gives a total projected cost of $4,727,790.10 for all repairs.

Mid-Bay Bridge Repair Costs
September 4, 2001

		Quantity	Unit	Cost	Status
Construction	Maintenance of Traffic	1	LS	$70,000.00	Complete
	Mobilizations	1	LS	$100,000.00	Complete
	Tendon Replacement	11	EA	$999,680.00	Complete
	Sheathing Wrapping	3640	LF	$145,600.00	Complete
	Anchorage Repair*	1	Cost Plus	$1,098,335.33	Complete
	Off-Duty Sheriff	1539	MH	$36,089.55	Complete
	Variable Message Sign	512	ED	$22,865.92	Complete
	Contingency	0.79	LS	$75,219.30	Complete
	Mock-Up Testing	1	LS	$40,071.35	Complete
	Granite Constructors Subtotal =			$2,587,861.45	
	Tendon Wrapping Contract**	87000	LF	$1,482,588.18	8/31/01 Bid
	Subtotal of Construction Costs =			$4,070,449.63	
Engineering	Construction Engineering Inspection (Emergency Repairs)			$534,474.47	Complete
	Bridge Evaluations			$99,469.00	Final Draft
	Duct Wrapping Design Documents			$23,397.00	Complete
	Engineering Subtotal =			$657,340.47	
	Total Cost =			**$4,727,790.10**	

* Portion of cost associated with vacuum grouting = $ 607,171.66
** Apparent low bid 8/31/01

Figure 7.1 – Cost Information for Repairs of the Mid-Bay Bridge Post-Tensioning System.

Chapter 8 – Precast I-Pier Investigations

8.1 Introduction

The vertical profile of the Mid-Bay Bridge is made up the following components:

- Begin Bridge to Station 208+48.72 – 0% grade at elevation 21.0
- Station 208+48.72 to Station 202+48.72 – Sag vertical curve with exiting grade of +3%
- Station 202+48.72 to Station 193+64.05 – Constant grade of +3%
- Station 193+64.05 to Station 169+79.05 – Crest vertical curve with exiting grade of -3%
- Station 169+79.05 to Station 160+94.38 – Constant Grade of -3%
- Station 160+94.38 to Station 154+94.38 – Sag vertical curve with exiting grade of 0%
- Station 154+94.38 to End Bridge – 0% grade at elevation 21.0

The piers for the low level spans are cast-in-place, reinforced concrete I-Piers resting on footings that are supported by precast prestressed piling. As the profile of the bridge rises to cross the navigational channel, the pier heights increase, and Piers 66 through 101 are not made of reinforced concrete but are precast segmental post-tensioned piers. Figure 8.1 shows a side and front elevation of a typical high-level pier for the Mid-Bay Bridge.

8.2 Inspections of I-Pier Post-tensioning Tendons

Inspections of selected piers were conducted to determine if voids existed in the anchors of the vertical tendons of the Mid-Bay Bridge post-tensioned I-Piers. Five piers were selected for inspection. Each pier had two holes drilled on their east face, one hole for each tendon on that face. Initially the intent was to drill into the tendons just below the confinement spiral, approximately 15” from the top of the piers. Heavy congestion of the reinforcing in the pier caps made these locations impractical, and the holes were drilled further down from the tops of the piers. If voids were found in the duct additional holes were drilled to determine the length of the void. The drilled holes were sealed with epoxy after inspection.

Pier	Dist. From Top of Cap	Condition
77 South Tendon	26”	Void, no moisture
77 South Tendon	29”	Void, no moisture
77 South Tendon	36”	Void, no moisture
77 South Tendon	43”	Void, no moisture
77 South Tendon	53”	No Void
77 North Tendon	26”	Could not inspect
77 North Tendon	53”	No Void
81 South Tendon	25”	No Void
81 North Tendon	25”	No Void
85 South Tendon	25”	No Void
85 North Tendon	25”	No Void
89 South Tendon	28”	No Void
89 North Tendon	29”	No Void
93 South Tendon	25”	No Void
93 North Tendon	25”	No Void

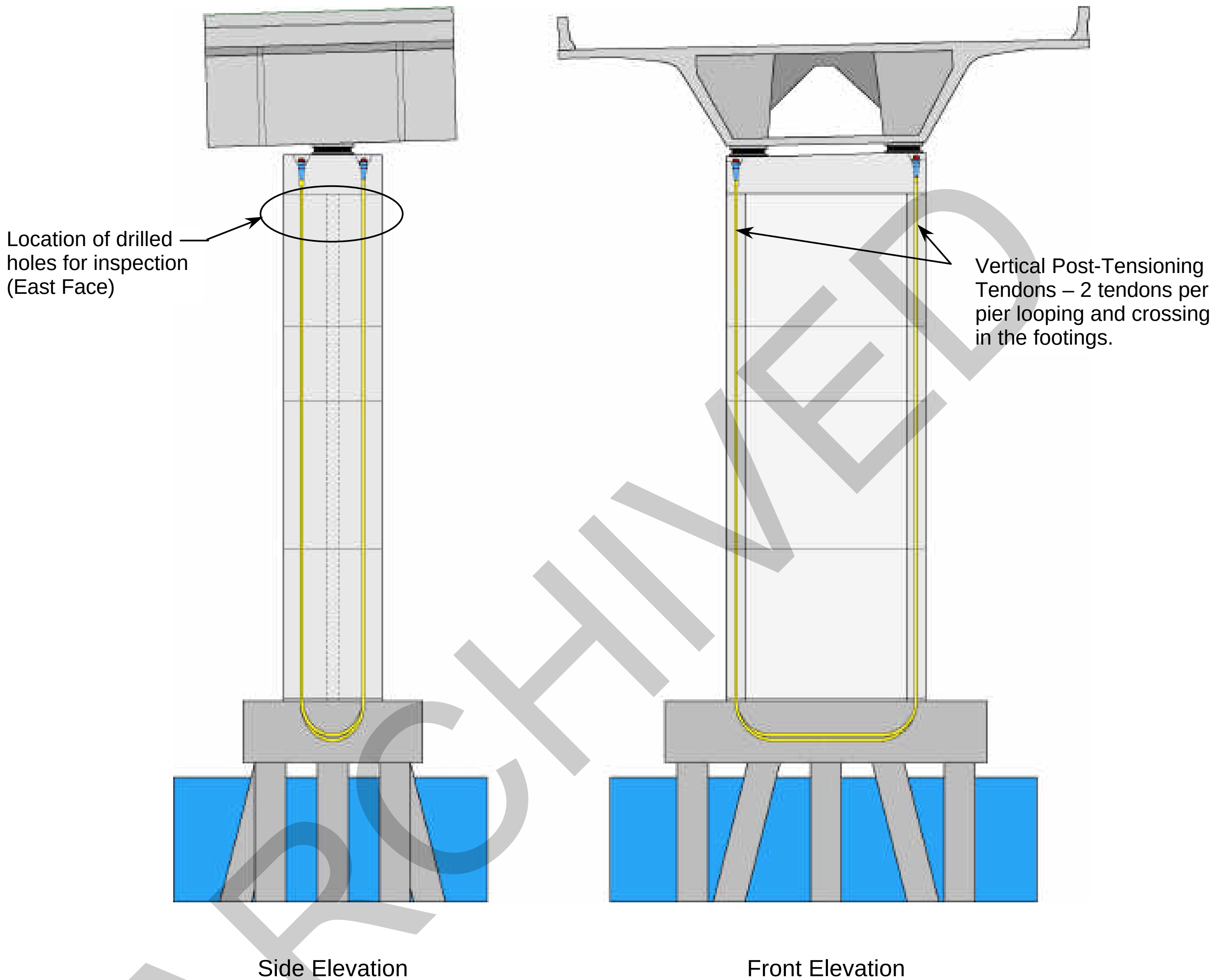


Figure 8.1 – Side and Front Elevations of Precast I-Piers

8.3 Ship Impact Analyses

Structural analyses were conducted to verify the ship impact of the Mid-Bay Bridge I-Piers with regard to the design values. Two approaches to the analyses were taken and the studies included reducing the area of post-tensioning steel to replicate loss of tendon cross section due to corrosion. Results of each of the analyses are presented in Appendix I of this report.

8.3.1 Original Design Methodology

The first analysis was conducted following the collapse mechanism method used during the original design of the bridge. This method independently determines failure capacities for each of the members resisting ship impact (piers, pile flexure, pile tension/compression, etc). Boundary conditions that may limit load in a member (superstructure uplift, bearing shear, etc) are then imposed. Next, equilibrium is checked by summing the moments of the individual shear capacities acting along their horizontal lines of action by the distance from the lines of action to the elevation of the applied impact force. Adjustments to the shear forces are made to obtain equilibrium and the final shear forces are summed and compared to the ship impact load.

The results of this analysis concluded that piers are capable of resisting the design criteria ship impact loads. Also, the capacity of the piers was not exceptionally sensitive to the level of corrosion assumed in the pier post-tensioning. In most cases, using this method, as much as 75 percent of the cross sectional area of the tendons at the base of the pier could be lost before a reduction in ship impact capacity was noted.

8.3.2 Refined Analysis

The second analysis was conducted using the Florida Pier computer program (version 3.1). This method uses a global stiffness solution with member forces and displacements related by their relative stiffnesses. The program also integrates material and geometric non-linearities in the solutions, and effectively models the soil/pile interaction within the solution.

The results of this second analysis also concluded that the piers are capable of resisting the design criteria ship impact loads. Different from the first analysis, however, this study concluded that complete development of the full tendon cross section was required at the base of the pier to resist the design loads.

Appendices will be added as made available.
Anticipated date is mid-December.