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US Army Corps of Engineers
Waterways Experiment Station

Durability of Posttensioned Concrete After 33 Years of Marine Exposure

*by Morris Schupack, Schupack Suarez Engineers, Inc.
Edward F. O'Neil, WES*

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by Morris Schupack

Schupack Suarez Engineers, Inc.
225 Wilson Avenue
Norwalk, CT 06854

Edward F. O'Neil

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

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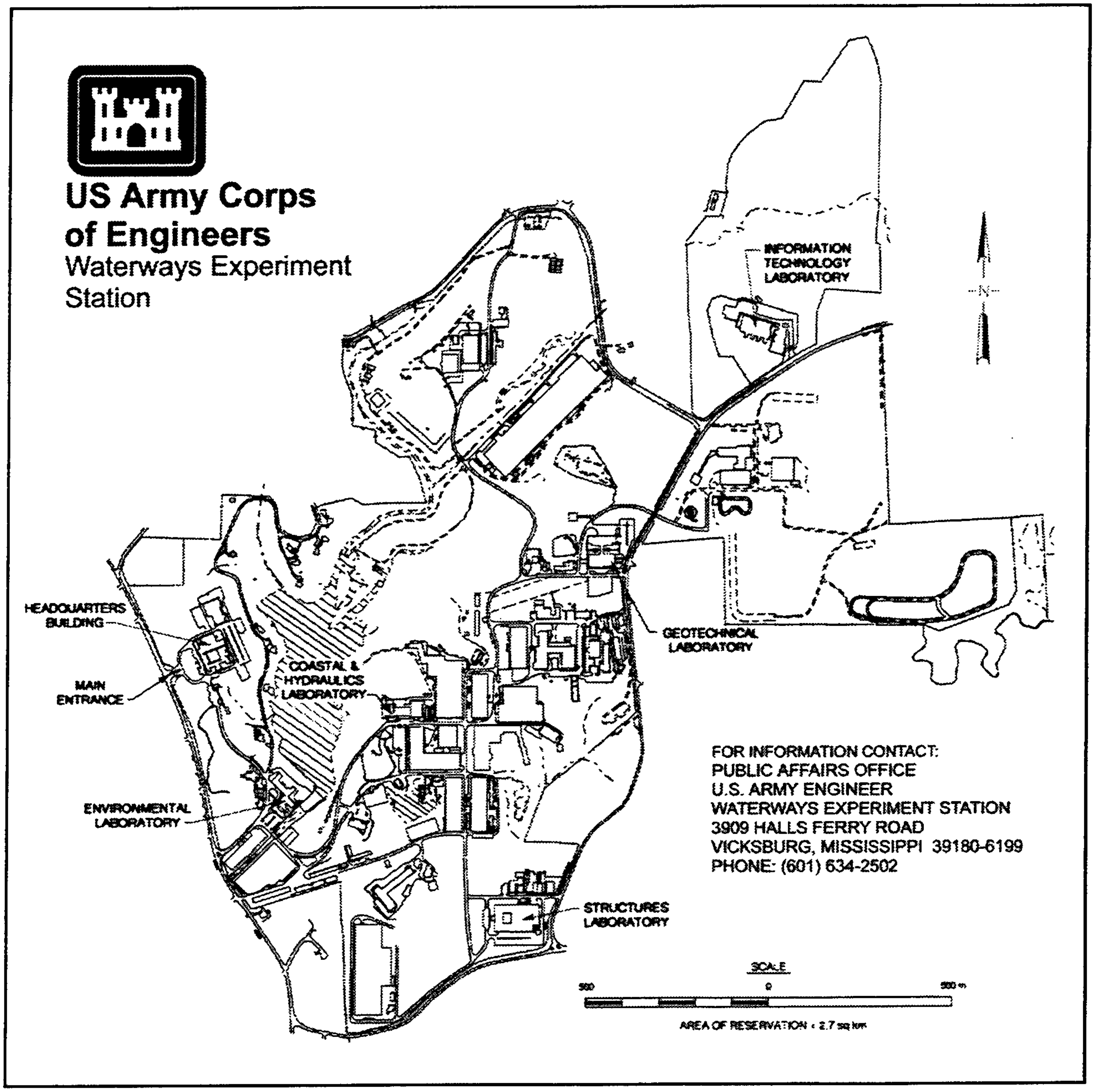

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PREFACE

The investigation of which this is one the reports forms a part of Engineering Study 031 (formerly Civil Works Investigation Item CW 031) and was authorized by multiple letter dated 11 December 1956 from Headquarters, U.S. Army Corps of Engineers, under a project plan entitled "Durability and Behavior of Prestressed Concrete Beams."

This report was prepared in part under a contract between the U.S. Army Engineer Waterways Experiment Station (WES) and the senior author as a joint effort between Schupack Suarez Engineers, Inc. (SSE), and the Engineering Mechanics Branch (EMB), Concrete and Materials Division (CMD), Structures Laboratory (SL), WES. The work was conducted under the direction of Dr. Bryant Mather, Director, SL, Drs. Tony C. Liu and Paul F. Mlakar, former Chief and Chief, CMD, respectively, and Mr. Edward F. O'Neil, Acting Chief, EMB. This report was written by Mr. Morris Schupack, SSE, and Mr. O'Neil. The WES Project Leader was Mr. O'Neil.

Director of WES during the preparation and publication of this report was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

1.0 INTRODUCTION.

1.1 The purpose of this report is to provide the history and results of the exposure tests of twenty post-tensioned beams installed in 1961 at the Corps of Engineers Severe Weather Exposure Station at Treat Island, Maine[1]. The report assembles the data and evaluates the findings. From this overall study, the important durability issues are reported, the decision on which additional beams to be autopsied are recommended and the best long term behavior features are identified. Details and materials which provide long term durability for similar construction are recommended.

2.0 HISTORY OF POST-TENSIONED CONCRETE BEAMS RESEARCH.

2.1 In the then newly developing technology of post-tensioning in the late 1950's, some behavior problems with end-anchor protection were encountered. This manifested itself by delamination of end-block cover or corrosion of tendon end anchorages, or both. In 1959, Schupack met with Professor Chester Siess, University of Illinois and Eric Erickson, then Chief Engineer for the Bureau of Public Roads to discuss the state-of-the-art regarding the protection of end anchors in post-tensioned bridges. Schupack was, at the time, involved in the design of several post-tensioned bridges and found a lack of knowledge existed regarding the best method of protecting end anchors. As an outgrowth of this meeting, an Reinforced Concrete Research Committee (RCRC) 6 Sub-Committee was organized consisting of Chairman C. F. Corns, E. L. Erickson, W. J. Jacobs, and M. Schupack. It was given the task of developing a test program that would provide knowledge in regard to post-tensioned anchorage protection.

2.2 Schupack was asked to design a research program to evaluate the durability of several types of end anchorage protection under severe natural weathering conditions and, after concurrence of the RCRC Committee, presented the test to the Corps of Engineers. After the Corps suggestions were included, the U.S. Army Engineer Waterways Experiment Station (WES) fabricated the twenty beams. They contained twelve different types of end-anchorage protection and four different types of post-tensioning tendons (Table 1). At that time, strand tendons were not used in post-tensioning; therefore, only wire and bar tendons were used in this program). The WES project "Durability and Behavior of Prestressed Concrete Beams" was begun in 1956. This post-tensioned phase was an extension of this.

2.3 The beams were fabricated and installed at mid-tide at the WES Severe Weather Exposure Station, Treat Island, Maine, during 1961. The Corps of Engineers performed annual inspections and made the site available for interested parties, biennially. Schupack attended a majority of the biennial inspections and conducted an independent survey of the beams documenting his findings by photographs and written notes. His field history of the durability performance supplemented by Corps data makes up the majority of the site observations available.

2.4 A progress report "Durability and Behavior of Prestressed Concrete Beams Report 2 - Post-Tensioned Concrete Investigation Progress to July 1966," Technical Report No. 6-570, dated March 1967[1], was prepared by WES.

2.5 In 1972, Schupack was asked to suggest the beams to be autopsied and how they should be investigated. A plan was agreed on between RCRC and the Corps of Engineers whereby eleven beams, eight in 1973-74, and three more in 1983, were selected and removed by the Corps of Engineers to WES for structural and material evaluation.

2.6 The first eight beams were failed, autopsied, and reported on in an extensive WES report, "Durability and Behavior of Prestressed Concrete Beams - Report 4 - Post-tensioned Concrete Beam Investigation with Laboratory Tests from June 1961 to September 1975," by Edward F. O'Neil, dated February 1977[2].

2.7 WES data collection was discontinued after 1979. Beams were maintained and photographed periodically by Dr. Ted Bremner of the University of New Brunswick under contract to WES.

2.8 In 1980, Schupack wrote, "The Behavior of Twenty Post-tensioned Test Beams Subject to up to 2200 Cycles of Freezing and Thawing in the Tidal Zone at Treat Island, Maine," published in ACI SP-65, August 1980[3].

2.9 In 1983, three other beams were removed for failure and autopsy and were reported on in WES Report 6, dated March 1984, "Durability and Behavior of Prestressed Concrete Beams; Post-tensioned Concrete Beam Investigation, Supplemental Tests of Beams Exposed from 1961 to 1982," by Edward F. O'Neil and Glenn L. Odom[4].

2.10 At the conclusion of this autopsy work, the Corps of Engineers wanted to abandon the research program. Schupack presented reasons why the study should not be abandoned and as a result, the remaining beams were maintained for inspection to date by interested parties.

2.11 Since 1982, Schupack made most biennial inspections and kept all his inspection data.

2.12 In 1994, the Corps of Engineers funded a Broad Agency Announcement Contract to Schupack to review the data to date and prepare this report.

3.0 CORPS OF ENGINEERS (WES) SEVERE WEATHER EXPOSURE STATION, TREAT ISLAND, MAINE.

The Severe Weather Exposure Station is located at Treat Island in Cobscook Bay near Eastport, Maine. This station, originally constructed to study concrete durability for the Passamaquoddy Tidal Power Project, has been in use since 1936 and is an ideal location for tidal exposure experiments, providing twice-daily tidal submergence of specimens and exposure to the effects of Maine's severe winters. Figure 1A shows an overview of the exposure site in 1968. The twenty post-tensioned beams are in the foreground. Figure 1B gives an overview of the exposure site in 1986. The platform in the background is an exposure rack set at midtide elevation. The nine remaining post-tensioned beams of the twenty inspected in 1961 are in the foreground. The specimens to the left of the post-tensioned beams are the abandoned reinforced concrete beams used in the "Tensile Crack Exposure Test - Report 4 Statistical Analysis of the Long-Term Durability of Series "B" Beams," H. T. Thornton Jr., March 1984, which was installed in 1954[5].

The post-tensioned beams are installed at mean-tide elevation on the beach rather than the exposure rack due to their size and mass and the alternating condition of immersion of the specimens in sea water, then exposure to freezing air, provides numerous cycles of freezing and thawing of the concrete during the winter. The average water temperature, which is 39 °F (3.9 °C) decreases, in general, autogenous healing and chemical reactions in the concrete. The tidal range is a mean of about 18 ft (5.5 m), with a maximum of about 28 ft (8.5 m) and a minimum of about 13 ft (4 m).

In winter, the combination of air and water temperatures creates an environment in which specimens at the mean-tide elevation are thawed to a temperature of about 37 °F (3 °C) when covered with water, and are frozen to temperatures as low as -9 °F (-23 °C) when exposed in the air. A recording thermometer, the bulb of which is embedded in the center of a concrete specimen, records these temperatures. A cycle of freezing and thawing consists of the reduction of the temperature at the center of a concrete specimen to below 28 °F (-2 °C), and its subsequent rise to about that figure.

The "Durability and Behavior of Prestressed Beams" program was begun in 1956. The first specimens were pretensioned beams that are reported on in WES Report No. 6-570 "Durability and Behavior of Prestressed Concrete Beams, Report 3, Laboratory Tests of Weathered Pretensioned Beams," by E. C. Roshore, October 1971[6].

The post-tensioned beams were placed at half-tide elevation at Treat Island, Maine in June 1961. The post-tensioned beams that have remained at the site from 1961 to 1994 (Fig. 2) have been subjected to 4002 cycles of freezing and thawing for an average of 121 cycles per year. This is an extremely severe exposure and, in comparing behavior with that in other environments, it should be put into proper perspective. For instance, the average annual freezing and thawing cycles in Montreal is about 75.

<u>Composition of Sea Water at Treat Island, Maine</u>

Constituent	Parts per Million (Sampled in 1959)
Total solids	35,275
Suspended solids	-
Dissolved solids	-
Calcium	370
Magnesium	1,175
Sodium	9,500
Potassium	370
Chloride	17,100
Sulfate	2,385

4.0 BEAM SPECIMEN DESIGNS.

4.1 Shape of Beam Specimens.

The original purpose of this program was to determine the behavior of different methods of protecting post-tensioning anchorages from corrosion. The study was originally designed primarily for this. To make the specimens as representative as possible of post-tensioning structures as known in 1959-60, the beam design was made in the shape of an "I" beam with rectangular end blocks (Fig. 3). Shaping the beams, using a 5 in. (127 mm) web, also made them lighter, thus facilitating handling at Treat Island.

Because of the small size of the "I" beam section and to control cracking, reinforcing was placed with a concrete cover in the flanges and webs of 3/4 in. (19 mm). This became an interesting resource to determine the effectiveness of such small cover in this hostile environment.

4.2 End-Anchorage Protection.

Because of the limited experience in post-tensioned construction in 1959-60, it was decided to design the study so that many types of end-anchorage protection could be evaluated. Two basic end-anchorage protection details were used as shown in Fig. 3 and 4.

- Exterior - anchorage placed on surface of concrete and protected by casting on a cover of concrete after post-tensioning (Fig. 4A to 4E).
- Flush - anchorage recessed into a pocket within the end of the beam, which is filled with protective material after post-tensioning (Fig. 4F).

The details of end-anchorage protection are listed in Table 1. The table is arranged in grouping of types of tendons, with beam number running consecutively as used in the fabrication and beam identification in the field. To facilitate evaluating anchorage protection, Table 2 reorganizes the anchorage protection by type as well as providing performance information with time. It groups the anchorage in flush and exterior types, with sub-divisions based on materials (Table 3) and details used as follows:

Flush - Portland-cement concrete.
Epoxy-resin concrete.
Portland-cement mortar.

Exterior- Portland-cement concrete with no reinforcement.
Portland-cement concrete with reinforcement.
Epoxy-resin concrete with no reinforcement.
Epoxy-resin concrete with reinforcement.

End-anchorage protection was attached to the beams using six different types of joint preparation:

- Bush-hammering.
- Epoxy-resin adhesive on sandblasted surface.
- Retarding agent.
- Sandblasted followed by primer.
- Sandblasted.
- No preparation.

This provided seven basic different types of end-anchorage protection, six different surface preparation methods and four types of post-tensioning systems which provided forty different end conditions. The variety provided the opportunity to observe the performance of different methods and materials and resulted in a better understanding of the behavior of various combinations of materials and details.

4.3 Post-tensioning Systems Used.

Nineteen of the beams used a single grouted tendon with bright steel flexible ducts, 1-1/4 in. (32 mm) and 1-5/8 in. (41 mm) O.D. The grout, used for bonding and protecting the prestressing steel, contained an aluminum powder expansion agent. Beam No. 13 contained one unbonded post-tensioning tendon which was grease-coated and spiral-wrapped with paper. The coating material consisted of non-volatile calcium soap and mineral oil grease with a rust-preventing additive. The eccentricity of the unbonded tendon or of the steel duct which contained the post-tensioning steel, was either 1 in. (25 mm), 2 in. (51 mm) or 3 in. (76 mm).

The types of post-tensioning tendons commercially available at the time were only those using bars or wires. At the initiation of this program, post-tensioning strand tendons were still in the developmental stage. This is the reason no strand tendons are present in this test. The tendon types and components used are shown in Table 4.

From Table 2 it can be seen that the tendon type and its anchorage details did not have any significant affect on the anchorage protection behavior.

4.4 Reinforcing Steel.

All beams contained normal reinforcing steel which was provided with a nominal 3/4-in. (19-mm) concrete cover. The details of reinforcement are shown in Fig. 3 and Table 4 of Appendix A. Figure 9-2G shows a typical reinforcing cage removed from an autopsied beam.

4.5 Mixtures Used to Fabricate Beam Specimens.

The mixtures used are summarized in Table 3. For more detailed information regarding mixtures and strength, refer to Appendix A, which is extracted from WES Report No. 6-570[2].

5.0 OBSERVATIONS OF BEAMS AT EXPOSURE SITE.

5.1 Chronology.

The following listing provides the chronology of the fabrication, beam exposure and observations that were made of the beams at the site until 1994.

Beams Fabricated	Sept. 1960 - March 1961
Placed at Treat Island	June 1961
Pulse Velocity and Rating by Corps	1961 to 1979
Data Gathering by Corps Discontinued	1980
Corps of Engineers Inspections	Every year until 1980
R.C.R.C. Committee	Biennially
Inspections by M. Schupack (biennially)	1968 to 1994
Beams Removed for Autopsy by WES:	
Five (5) Beams	Sept. 1973
Three (3) Beams	Dec. 1974
Three (3) Beams	Jan. 1983
Beam Autopsies at WES	1973, 1977, 1983 & 1984
Autopsy Reports by WES	Feb. 1977, Oct. 1984

5.2 Visual Inspections.

Visual inspections were made by WES, University of New Brunswick, RCRC, M. Schupack and other interested parties over the course of the 33 year exposure period. Available data from these inspections was reviewed and information useful to this report included herein. The post-tensioned beams were visually inspected in the summers of 1962, 1963, 1964, and 1966 by a panel of qualified observers. The beams were rated by the observers in accordance with a set of instructions[2]. The condition of each beam, based on the inspection notation made by the observers, was expressed numerically, using a scoring system. As the beams deteriorated, it was found that this scoring system was difficult to interpret and the procedure was discontinued probably after 1966. Schupack made 10 site inspections. The beams were sounded and visually inspected. Observations were dictated and transcribed, notes were made on tabular recording sheets and photographs taken.

5.3 Pulse-Velocity Tests.

Pulse-velocity measurements were made by WES on all beams prior to their installation at the Treat Island Exposure Station. These measurements were made in accordance with applicable provisions of test method CRD-C 51-57. Additional pulse velocity measurements were taken by WES annually until 1980. The readings were taken by transmitting an ultrasonic pulse through the beam along its long axis (longitudinally) and also through the thin web section (transversely). The pulse velocity instrument measures the time of travel of the sound pulse, and from the time of travel and the path length, values for the velocity of sound in the concrete (V) are obtained. The square of the pulse velocity, V^2, is used in determining the change in durability because the velocity is proportional to the square root of the modulus of elasticity divided by the density. Therefore the modulus of elasticity is proportional to the square of the velocity, V^2. The square of the velocity thus determined is expressed as a percentage of the square of the initial velocity obtained at installation ($\%V^2$). The change in $\%V^2$ is used in evaluating deterioration.

The transverse pulse velocity values of V^2 obtained during the exposure were erratic and did not provide a satisfactory indication of the changing condition of the beams. This is primarily due to the short length of pulse travel (5 in. (127 mm)) and the type of pulse velocity equipment used during the 1960's and 70's. The longitudinal pulse velocity values ($\%V^2$) appear to be reasonably consistent and did indicate some deterioration in the beams since eight of the 1965 longitudinal ($\%V^2$) readings are below 100%.

6.0 OVERALL PERFORMANCE OF POST-TENSIONED BEAM SPECIMENS.

The performance of the post-tensioned beams is related to the durability of details and materials and their interaction in the Treat Island environment. To describe critical observations it will be attempted to discuss key issues separately. Since the interaction of different materials and details may affect adjoining elements, the discussion of a particular issue may result in repetition. The following detailed description of distressed elements should provide design guidance.

The post-tensioning beams in the severe Treat Island environment did not suffer overall structural failures. The degradation that occurred only nominally affected the load carrying capacity. Figure 2 shows the beams at low tide on their supports in 1962, 1972, 1982, 1990, and 1994. The eight beams autopsied in 1973 and the three in 1984 did not reveal any flexural cracking or a decrease in expected structural capacity when tested to failure. The remaining nine beams in 1994 did not show any indication of tendon failure or flexural cracking. No evidence of tendon failure occurred even though end-anchorage protection failures occurred as early as 1963. The 1973-74 autopsy of eight beams and the 1984 autopsy of three beams did not reveal any tendon failures.

To illustrate the degree of deterioration with time of the post-tensioning beams, a photo history of Beam 2, 5, 12, and 16 are shown in Fig. 5, 6, 7 and 8.

In reviewing a particular beam, it is suggested that the photos be chronologically reviewed, then choose a specific detail to evaluate and then check this detail photo by photo.

6.1 End-Anchorage Protection.

6.1.1 The performance of end-anchorage protection, as judged from biennial inspections, is shown in Table 2. To compare the behavior of flush and exterior protection and variations within these methods, the table has been organized to group flush and exterior in major sub-categories. Table 2 has been condensed in Table 5 to more conveniently identify the condition at time of autopsy and in 1994. It also dates the year of the anchorage protection failure. The anchorage protection failure is defined as when seawater, by visual observation, apparently has access to the anchorage hardware. These observations are primarily based on Schupacks' judgment, notes, and photographs.

6.1.2 The obvious purpose of the anchorage protection is to prevent corrosion of the anchorage hardware, the prestressing steel penetrating the anchorage and the prestressing steel extending beyond the anchorage. Experience has shown that often when end anchorage corrosion occurs, progressive corrosion of the prestressing steel is likely to occur. To prevent this corrosion it is necessary to have protection, particularly in a chloride environment, which is highly resistant to chloride ingress requiring all "cold joints" to be water tight for the life of the structure.

6.1.3 Summary of end-anchorage protection longevity (Refer to Table 5). (Where photos are available, they are referenced below.)

6.1.3.1 Flush-anchorage protection.

A Portland-cement concrete.

1. Surface preparation - sandblasted, followed by epoxy-resin adhesive, 2 ends (Fig. 6B).

   a) Both are still in excellent condition.

2. Surface preparation - none, 2 ends.

   a) One failed in 1978.
   b) Other autopsied in 1974, excellent condition.

B. Epoxy-resin concrete.

1. Surface preparation - sandblasted, followed by primer, 2 ends.

   a) One failed in 1972, autopsied in 1983 (Fig. 10-1A).

b) Other autopsied in 1974, excellent condition.

C. Portland-cement mortar.

1. Surface preparation - sandblasted, 2 ends.

a) One autopsied in 1974, excellent condition.
b) Other failed in 1994.

6.1.3.2 Exterior portland-cement concrete.

A. No reinforcement through cold joint.

1. Surface preparation - retarding agent, 4 ends.

a) Two failed in 1972, autopsied in 1973.
b) Other two autopsied in 1973-74, good and very good condition.

2. Surface preparation - sandblasted, followed by epoxy-resin adhesive, 4 ends.

a) All four failed by 1978.

3. Surface preparation - bush hammer, 4 ends.

a) Two failed by 1972.
b) Other two autopsied 1973, good condition.

4. Surface preparation - none, 4 ends (Fig. 9-1A).

a) Two failed by 1972.
b) Other two autopsied 1973, poor and very good condition.

B. With reinforcement through cold joint.

1. Surface preparation - bush hammer, 4 ends (Fig. 5-3K).

a) Two failed in 1978.
b) Other two in good condition as of 1994.

2. Surface preparation - none, 4 ends.

a) Two failed.
   - one in 1978.
   - one in 1994.
b) Two in good condition as of 1994.

6.1.3.3 Exterior epoxy concrete.

A. No reinforcement through cold joint (Fig. 8-2G).

1. Surface preparation - sandblasted, followed by primer - 4 ends.

a) Three failed by 1978.
b) One autopsied in 1974, good condition.

B. With reinforcement through cold joint (Fig. 7-3K).

1. Surface preparation - sandblasted, followed by primer - 4 ends.

a) Three failed by 1982.
b) One autopsied in 1974, good condition.

6.1.4 The listing below rates the anchorage protection types which offered the best overall protection of beams still at Treat Island.

END ANCHORAGE PROTECTION TYPE	BEAM	SURFACE PREPARATION	CONDITION IN 1994 OR YEAR FAILED	COMMENTS	FIGURES
Flush concrete anchor protection	5S	Sand blast Epoxy-resin adhesive	Excellent	Excellent condition.	6A & B
Flush mortar anchor protection	12S	Sand blast	1994	End started to fail 1990 Totally failed 1994	7-1A,B,C,D
Exterior concrete anchor protection with reinforcement	2L	Bush hammer	Good	Ends show rebar corrosion and spalling due to low cover, however anchor protecting still seems to be protecting the anchor.	5-1,2,3
Exterior concrete anchor protection with reinforcement	2S	None	Good		

6.1.5 Deterioration of Concrete Associated with Exterior Epoxy-Resin Concrete Anchorage Protection.

In all cases, the epoxy-resin concrete itself showed no apparent deterioration for all of the specimens to date. In general, the edges are still sharp and no visible surface deterioration was evident.

Although the epoxy-resin itself behaved very well for 33 years, the interaction between the epoxy-resin and the concrete manifested deterioration of the concrete interface and corrosion of the reinforcing steel that extended from the portland-cement concrete into the epoxy-resin concrete (Fig. 8-1, 8-2, and 8-3I). The concrete deterioration first appeared in 1972 in beam ends 16L and 20S. (Table 2-3). Prior to 1972 it appeared that the epoxy-resin concrete was a superior detail. The concrete deterioration was probably affected by the following conditions:

- Difference in coefficient of thermal expansion of the epoxy-resin concrete and the portland-cement concrete contributed to the failure of the interface bond between epoxy resin and concrete. Work performed by WES[7] indicates that the coefficient of expansion of the concrete in the beam was about $5 \times 10^{-6}/°F$ ($2.8 \times 10^{-6}/°C$) and the coefficient of expansion of the epoxy-resin concrete end protection was, $19.8 \times 10^{-6}/°F$ ($35.64 \times 10^{-6}/°C$) for sea end and $6.2 \times 10^{-6}/°F$ ($11.16 \times 10^{-6}/°C$) for the land end. Also, because of the differences in the rate of thermal movement due to the differences of thermal response to temperature change for both materials, the interaction is further exacerbated. The increasing modulus of elasticity of epoxy-resin concrete as temperature decreases[8] also contributes to greater interfacial stresses due to difference of coefficient of expansion.

- The epoxy-resin concrete is practically an impervious material as far as water penetration is concerned. Concrete on the other hand will absorb water and if it is critically saturated when subjected to freezing temperatures, the concrete will experience deterioration if it is not adequately air-entrained.

Even though the above cited reasons are likely explanations for the problem of interfacial failure of the portland-cement concrete contiguous to the epoxy-resin concrete, it was observed that this did not necessarily occur across the entire depth of the beam.

Once an initial gap is created by interfacial concrete deterioration, additional deterioration between the beam and the cap could be aided by water freezing in the opening and expanding. This is a condition that further opens the crack allowing more water to collect and freeze on the next cycle of freezing and thawing and further deteriorate the interface. If reinforcing steel passes through the interface, as soon as water has access to the steel not protected by portland-cement paste, corrosion will occur and the consequent expansive products of corrosion further deteriorate the surrounding concrete. The concrete deterioration process then makes water available to the anchorages with resulting corrosion.

Beam 4, land end, had an epoxy-resin concrete exterior protection. It was in good condition in 1983 when it was autopsied. Figure 10-1B shows the Freyssinet anchorage after removal of the epoxy-resin concrete.

The epoxy-resin concrete exterior anchorage protection appeared to be the best protection for about the first 10 years. After that, progressive deterioration at the interface was noted and shortly thereafter the reinforcing steel passing through the joint showed corrosion.

6.2 Concrete Durability.

The air-entrained concrete both in the beam proper and the end-anchorage protection behaved well in providing resistance to freezing and thawing. The 0.45 w/c - 6 bag (335 kg/m^3) cement content mixture in the beam concrete suffered less surface erosion than the 0.8 w/c - 4 bag (225 kg/m^3) cement mixture in the cast-on exterior-anchorage protection. The photo history of Beam 2, Fig. 5, particularly shows the difference in behavior in Photos C, E, F, J, and K. Industry experience generally shows that the higher the w/c the more difficult it is to obtain concrete which is resistant to freezing and thawing. The relatively good behavior of the 0.8 w/c mixture in the Treat Island environment suggests that is the air-void system is proper then the high w/c concrete can be resistant to freezing and thawing.

6.3 Reinforcing Steel Corrosion.

The corrosion of the reinforcing steel occurred randomly and is generally associated with locations where there was 3/4 in. (19 mm) or less concrete cover. The progression of rebar corrosion can be seen in Fig. 9, photo history of Beam 17, which was autopsied in 1983. The consequence of rebar corrosion, spalling of the surrounding concrete is shown in Fig. 9B, C, D, and F. The cage of reinforcement removed after autopsy of this particular beam is shown in Fig. 9G. Note that the extent of the corrosion is limited in the twenty two year exposure. In most cases where reinforcing bars are exposed by spalling of the surrounding concrete, the intensity of corrosion, even after 33 years of exposure, was not great. In most cases the bar deformations were still evident in 1994. This may be because of the relatively cold temperatures at Treat Island.

Figure 9D shows one side of the top flange spalled and the other flange in excellent condition. Possible reasons why some areas surrounding reinforcing bars did not spall are that the consolidation of the concrete in this area was more complete, the cover was more than specified, and the curing was locally superior in these sections.

Photo history of Beam 2, Fig. 5 also shows the progressive signs of reinforcing bars corrosion. Note that the exterior end-anchorage protection, on the sea (S) end showed reinforcing bars corrosion in 1978. This end protection performed well and protected the anchorage probably up to 1994 when spalling caused by corrosion may have permitted sea water to reach the anchorage. It is evident that if the reinforcing bars corrosion could have been prevented, this exterior anchorage protection could have resulted in a much longer service life.

It is evident that the web reinforcement passing through the 5-in. (127-mm) web suffered generally the least amount of bar corrosion although it also has 3/4 in. (19 mm) of cover. No web reinforcement corrosion was evident on the surface from any of the beams autopsied in the 1973 or 1983 studies. As can be seen in Fig. 8I through 8L, the first visually identified indications of web reinforcement corrosion was in 1984, in 1994, web corrosion was noted in at least two beams. If any other beams are autopsied it would be advisable to do a careful half-cell monitoring of all the reinforcing steel to determine the extent of corrosion activity as well as other types of corrosion testing.

In the WES Report[2] referring to the 1973-74 beam autopsy, it is reported that the reinforcement cages extracted from the beams indicated only mild corrosion, with the severest corrosion occurring at welds. The WES Report[4], referring to 1983 autopsy, indicates the magnitude of the corrosion on the stirrups was no greater than that observed on the stirrups in the 1975 autopsy.

In the 1973-74 autopsies as reported in the WES Report[2], the chloride gradient through the webs indicated sufficient chlorides to expect that all reinforcement would have been seriously corroded. Figure 11, was extracted from the WES Report and modified to show the chloride threshold at which there is a 90% chance that corrosion will occur. It is surprising that with this amount of chloride contamination found after twelve years, that the remaining nine 33 year old beams show very limited evidence of web stirrup corrosion.

The only complete wasting of reinforcing steel that has occurred are those that were contiguous to and in epoxy-resin concrete. Figure 7G through 7L and 8B through 8G shows similar performance. The rebars penetrating the epoxy were destroyed to an unknown depth into the end cap and the bar adjacent to the epoxy-resin concrete interface also was completely corroded and had spalled the concrete off. The reason for this type of complete dissolution of the reinforcing steel is not fully understood. A possible explanation for this is the far superior electrical isolation of the epoxy concrete compared to the portland-cement concrete and the consequent large cathode-small anode corrosion cell. It is also possible that once the interface between the epoxy resin and the concrete suffered concrete deterioration, the reinforcing steel progressively corroded inward toward the body of the epoxy resin. A further study of some of the remaining beams may reveal the reasons for this more aggressive corrosion and the progression of the rebar corrosion into the epoxy-resin concrete.

6.4 Post-tensioning Tendons.

6.4.1 General.

In the 33-year history of the nine remaining beams, no indication of tendon corrosion or failure appeared. The telltale mark of well grouted bonded tendon corrosion is usually a crack following the trajectory of the tendon[13]. This can also occur from water freezing in poorly grouted tendons. The 1973 and 1983 autopsies showed excellent grout filling. Careful inspection of the beams during field visits and the WES autopsies did not reveal any outward signs of tendon corrosion between the anchorages. Because of anchorage protection failures, prestressing steel projecting beyond the anchorages and within the anchorages, had minimum to severe corrosion.

The effectiveness of a well grouted tendon is demonstrated by the fact that no tendon failed even though:

- Failure of some anchorage protection occurred as early as in 1963. The first photographic record of an anchorage failure was in 1972. Table 5 shows that anchorage protection failures occurred throughout the history of the exposure study.

- The chloride ion level in the test beams webs enclosing the tendons were as high as 2,000 ppm, well above the level at which corrosion usually occurs.

Of the eleven beams autopsied in 1973, 1974, and 1983, no significant corrosion of the prestressing steel between the anchors was found except for the unbonded wire tendon which showed a small decrease in ultimate strength as well as its ductility. The WES Autopsy Reports[2,4] describe in detail the condition of the tendons. They utilized relative terms to describe the intensity of the corrosion observed. The terms heavy, moderate, and light were used. For the bonded tendons no reduction in ultimate strength of the prestressing steel occurred. The use of the terminology of heavy, moderate, and light was comparative for the conditions found. Heavy corrosion can be considered superficial since no loss of wire strength was reported. Quoting from Ref. 2 (O'Neil Report No. 4, page 17):

> "The strands (prestressing steel) in this study that were labeled as heavily corroded did not have deep corrosion and deterioration of the metal, with the exception of the landward end of Beam 1B. The rest of the strands had not lost more than 0.005 in. (0.13 mm) in diameter when they were cleaned and sanded. In general, when the classification heavy was used to describe the strand of steel, the percentage of surface area covered was less than or just at 100 percent and corrosion deep in the strand was not present."

6.4.2 Prestressing Steel.

The degree of corrosion between anchorages of the bonded prestressing steel found in the 1973-74 autopsy was comparable to that found in the 1983 autopsy. Information is not available as to the condition of the prestressing steel before they were placed into the forms, and after the concrete was cast and the beams moist cured for several weeks prior to grouting. The complete tendons were fabricated at the tendon suppliers plant, possibly 3 or 4 months prior to being protected by portland-cement grout. Because the sheath and trumpets masked the prestressing steel, WES could not observe the extent of prestressing steel corrosion at beam fabrication. From experience in the field it is known that wire exposed to a humid atmosphere will corrode. The visible corrosion can take place within a few days of exposure[11]. Usually the location of heavier corrosion is between the contact lines of wires or where the prestressing steel contacts the metallic sheath. It is presumed that since there was no progress of the corrosion process comparatively between 1973, 1974, and 1983, that corrosion process was abated in the environment it was in.

Considering the high chloride content of the webs (Fig. 11), it would have been expected that corrosion of the sheath and eventually the enclosed prestressed steel would have been advanced. If additional beams are autopsied and it is found that the corrosion process has not progressed, then it can be speculated that the corrosion present on the prestressing steel wires may have occurred during the early stages of fabrication of the tendons and the beams. It appears at this time, that the portland-cement grout protecting the prestressing steel has afforded excellent protection.

A summary of WES findings regarding the corrosion of the prestressing steel from Refs. 2 and 4 are included as Appendix B.

6.4.3 Steel Sheath.

The corrosion of the bright steel sheath was erratic and may have been partly present when the beams were fabricated. This was possible due to the nature of the tendons used. After the tendons are fabricated, it is physically impossible to examine the inside of the sheaths for corrosion sites. Even if the tendons were carefully stored prior to fabrication of the beams, during beam fabrication and curing, the wires and sheath will be exposed to water and air and until they are grouted this situation will continue. Figure 9-3I-J-K shows some of the typical corrosion found at the 1973 and 1983 autopsies. A few instances where the corrosion penetrated the sheath was found. With the high chloride ion content in the webs, this should be more widespread based on experience in other environments.

6.4.4 Reasons for Limited Corrosion.

The possible reasons for the limited corrosion of the sheath and the prestressing steel are:

- Concrete was permanently saturated because of the twice daily inundations and consequently oxygen was not available for the corrosion process.

- Magnesium carbonate may have precipitated in the void system, decreasing the concrete permeability and thus reducing the availability of oxygen.

- The relatively low temperature of the sea water (average 3.9 °F (39 °C)) slows down the electro-chemical process of corrosion.

- The web concrete may have been very well consolidated by internal vibrators in a 5-in. (127-mm) web producing a low-permeability concrete.

6.4.5 Grout - 18 Beams Neat Cement Grout, 1 Beam Sanded Grout.

The portland-cement grout mixtures are shown in Table 3. The cement grout provided excellent filling of the sheath and corrosion protection.

The steel tendon in nineteen of the twenty beams was pressure-grouted when the concrete was 17 to 40 days old. Eighteen of the beams were grouted with neat cement grout; the other was a natural-sand grout (Beam 14). All grouts contained a small amount of aluminum powder to produce grout expansion. A complete description of the grout and the time intervals involved is reported in Reference 1. Composition of the grout is shown in Table 3. The total chlorides from samples tested (Ref. 2) were less than 200 ppm by mass of grout. This indicated only trace amounts of chlorides within the grout while immediately adjacent to the outside of the sheath the amount was as high as 1,900 ppm. Chlorides in concrete immediately surrounding the sheath were generally in the 300 - 1,800 ppm.

The grout generally seemed to fill the tendon well although there were some settlement voids. The protection the grout offered seemed to be excellent. There is still some question about when the light corrosion of the prestressing steel, which was random, occurred. Since corrosion of the prestressing steel and the sheath were comparable in 1973, 1974, and 1983 autopsy, it is assumed that the grout afforded excellent protection to the prestressing steel.

6.4.6 Unbonded Tendon - Grease and Paper Wrapped.

Beam 13, autopsied in 1974, contained an unbonded 1/4-in. (6.4-mm) diameter-8 wire button headed tendon. This tendon was coated with grease and then spirally wrapped with a heavy paper to form a sheath. Both end exterior anchorage protections, as received by WES, had developed cracks between the caps and the beam. The exhumed tendon hardware and the wire button heads showed various degrees of corrosion. This indicated that water has reached the anchorage and, therefore, the anchorage protection had failed.

If water had access to the anchorage hardware it is most likely it had access to the rest of the prestressing steel for the greased and paper wrapped tendon. Experience has shown that if water has access to the anchorage, it will usually find its way into the body of a grease-protected tendon if grease does not completely fill the void system[12].

The autopsy of the tendon revealed that the sheath was composed of heavy paper, spirally wrapped in several layers that covered the grease coated prestressing wires. The paper wrapping was found to be heavily damaged. In at least five places the paper was torn with two places exposing grease and the wires. The remainder of the sheath was frayed and tattered. It was determined that the damage was not from the concrete removal because portions of the paper had become thin and brittle with age. At two spots where wires were exposed, they were covered with concrete dust that had stuck to the grease. For the most part, the wires in these areas were dry and at least 50% of the exposed area was not covered with grease.

When the paper sheath was removed, the wires were stained black from the grease. In some places, the grease still coated the wires and was sticky. In other placed, the wires were black, covered with a dried-out grease or exhibited no grease. The places where the grease was dried up or absent, corrosion occurred.

One wire was found failed near the anchorage. It is presumed that this occurred in the flexural test prior to autopsy. In individual tests of the wires the most corroded wire had an ultimate strength of 239,222 psi (1,650 MPa) and an elongation of 1.65%. This was lower than the ASTM specification requirement of 240,000 psi (1,655 MPa) and 4%. All other wires met the ultimate strength but had elongations of 2.62 to 3.5%. The paper-wrapped sheath and the coverage and quality of the grease protection was inadequate and was the direct cause of the decrease in the mechanical properties of the wire.

7.0 COMPARISON OF PERFORMANCE OF POST-TENSIONED BEAM SPECIMENS WITH COMPARABLE FIELD FINDINGS ON STRUCTURES

7.1 Epoxy-Resin Concrete Composite with Precast Concrete.

Railroad platforms in the Northeast U.S.A. used an epoxy-mortar curb to retain a bituminous wearing coarse overlay over precast sections. Details used are shown in Fig. 12-A. The long-term effects of the interaction of the epoxy resin and.the concrete are also shown in Fig. 12-B and 12-C. The degradation of the concrete at the interface and below, is similar to that experienced in Beam 12, land side as can be seen in Fig. 7-2 and 7-3. Figure 12-B and 12-C show photographs of some of this type of action. The thermal incapability caused the epoxy-resin mortar curb to crack. This crack propagated into the concrete and the resulting distress occurred with actual concrete deterioration in the air-entrained concrete.

7.2 Flush Anchorages.

Flush anchorages are typically used on single-strand unbonded-tendon systems. The practice has shown that if the recess is properly cleaned and packed with preferably a "non-shrink mortar," that excellent performance occurs. Some inadequate preparation of surface and porous packing mortar has caused corrosion of anchors and permitted access of water to the tendons. Figure 13-A and 13-B show defective end-anchor pockets.

7.3 Add-On Anchorage Protection.

The importance of reinforcement passing through the cold joint of add-on anchorage protection can also be seen in Fig. 14. The end-anchor protection for this 35-year old post-tensioned bridge in the Northeast United States indicates the action occurring at the cold joint between main concrete and the add-on anchorage protection. More reinforcing through this cold joint might have avoided some of this distress as well as better preparation of the bonding surface. The add-on concrete was placed without any bonding compound. As can be seen in Fig. 14, other anchors do not suffer the same degree of distress. It is important to point out that an open bridge joint existed at this pier location and the salt-laden water was free to fall through the joint and onto the end-anchor protection. This is generally no longer an accepted practice.

8.0 CONCLUSIONS.

8.1 General.

This 33-year experience of observing the performance of post-tensioned precast relatively thinly proportioned beams at Treat Island was an extreme test of the durability of post-tensioning systems. It can be stated that if post-tensioning follows the accepted good practices of today, it can withstand the harsh environment of Treat Island or less severe field conditions.

8.2 End-Anchorage Protection.

The main original purpose of the post-tensioned beam study was to determine the most effective end-anchorage protection. Results indicate that the flush anchor packed with air-entrained concrete with epoxy-resin bonding agent, was the most durable protection but it requires complete bond to concrete. The next best anchorage protection was the exterior anchor which had reinforcement passing from the parent concrete into the add-on concrete. The exterior anchorage showed failure because of reinforcement corrosion, which unfortunately had only 3/4 in. (19 mm) of cover. With 2-1/2 in. (63.5 mm) to 3 in. (76 mm) of cover, these anchorages would probably not have failed.

The exterior epoxy-resin concrete showed itself to be a superior system for the first 10 years. After that, concrete deterioration at the concrete to epoxy resin interface shows this as not being an acceptable system for this environment.

8.3 Tendon Corrosion.

Anchorages - The anchorages suffered various degrees of corrosion due to anchorage-protection failures. Some anchorage protection failed in the early 60's, less than 1 year after installation.

No anchorage corrosion occurred which caused the tendon to fail. Anchorages which were exposed for up to 26 years had surprisingly small amounts of corrosion.

Sheaths - The corrosion of the steel sheaths for grouted tendons was erratic and generally seemed to be a minimal problem considering the chloride content of the beams and the environment.

Bonded Prestressing Steel - Prestressing steel corrosion between anchorages, based on the autopsy, indicates very little loss in strength with the bonded steel still maintaining its ultimate strength. It appears that the corrosion found in the 1973 autopsy did not progress in the 1983 autopsies. Therefore, either the corrosion was abated in this environment or the corrosion was present when the tendons were grouted.

Unbonded Prestressing Steel - The greased and paper-wrapped unbonded tendon indicated more corrosion than the bonded tendons. The one unbonded tendon had a wire fail during the flexural strength test. In evaluations of individual wires, one failed below the required ultimate strength and all wires lost some of their ductility. For this environment, the unbonded tendon using paper wrapping is unsatisfactory. The use of an extruded plastic sheath would be adequate with proper end anchorage protection.

8.4 Concrete Durability.

The durability of the beam concrete, considering the harsh environment and the corrosion of the reinforcing steel with small cover, was excellent.

The importance of the cold-joint treatment is shown by the varied behavior of the end anchorage protection. This shows the need to carefully detail and reinforce the cold joint interface.

8.5 Reinforcing Steel Behavior.

The performance of the reinforcing steel with only 3/4 in. (19 mm) cover was better than anticipated. There are portions of the beam that have 3/4 in. (19 mm) cover to the reinforcing steel which have not spalled at this time. Most of the flanges indicate some spalling. It is evident, however, that 3/4 in. (19 mm) cover in this environment is not acceptable.

9.0 RECOMMENDATIONS.

9.1 Additional Autopsies.

It is suggested that Beams 2 and 16 be removed to WES and autopsied. The procedure used in the 1973 and the 1983 autopsy does not necessarily have to be followed. It is suggested that the following be performed.

- Do a non-destructive corrosion evaluation of the prestressing steel and the reinforcing steel using half cells.

- Perform chloride profiles through web adjacent to sheath and in areas of anchors. Relate these results to corrosion potentials.

- Perform petrographic evaluation of the concrete in both the add-on concrete and the parent concrete to determine if magnesium carbonate has filled any of the voids.*

- Determine the degree of saturation.

- Determine the permeability of the concrete.

- Remove the tendon, with the care taken in the previous autopsies, and examine the components for corrosion. Compare the corrosion of the sheaths and the wires with the previous autopsies.

9.2 Continued Observations of the Performance of Post-tensioning Tendons.

The long-term performance of the post-tensioning tendons is of interest. It is therefore suggested that the remaining beams be maintained for possible future evaluation of the post-tensioning systems.

9.3 Performance of Reinforcement in Epoxy-Resin Concrete.

Make a study of the reinforcing in epoxy-resin concrete to explain reinforcement corrosion and epoxy-resin end-block protection.

10.0 ACKNOWLEDGMENT.

The long-term involvement of the Reinforced Concrete Research Council (RCRC), which was the main driving force to initiate this program, and the long-term involvement of the Corps of Engineers in maintaining the exposure site and recording data is appreciated. The assistance and writing of this report and the effort of organizing the data and tabulating it into more easily evaluated data was done by Dan Schupack of Schupack Suarez Engineers (SSE) Inc. The recognition by Dr. Tony C. Liu of the Corps of Engineers, of the importance of bringing to the industry the experience of the 33-year exposure tests of the post-tensioned beams is a service to the profession.

* An estimated air void cannot become filled (or partially filled) with solids precipitated from solution on drying since any water entering such a void during freezing must leave abruptly as thawing takes place so the void will be empty (air filled by decompressed air) at the next freezing. Any void with precipitated solids in it ceased to be a functioning entrained air void; i.e., it was transected by a crack before water with dissolved solids could stay in it when the concrete was thawed.

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P.T. System	Beam No	Eccentricity of Tendon (inches)	Eccentricity of Tendon (mm)	Estimated Final P.T. Force (tons)	Estimated Final P.T. Force (kN)	Land or Sea End	Details of End Protection: Flush or Ext.	Mat'l	Cement Factor Bags/ cu. yd.	Cement Factor Bags/ cu. meter	Water/ Cement Ratio	Slump (inch)	Slump (mm)	Air Con-tent (%)	Compressive Strength 28 days (psi)	Compressive Strength 28 days (MPa)	Reinf. Steel	Surface Prep.
Freyssinet 12 - 5 mm diam. (A)	1*	0	0	23	205	L	Flush	Conc.	3.94	4.39	0.80	1.5	38	4.0	3,680	26	No	None
	1*	0	0	23	205	S	Ext.	Conc.	3.94	4.39	0.80	1.5	38	4.0	3,680	26	No	Retarding Agent
	2	0	0	23	205	L	Ext.	Conc.	3.94	4.39	0.80	1.5	38	4.0	3,410	24	Yes	Bush Hammer
	2	0	0	23	205	S	Ext.	Conc.	3.94	4.39	0.80	1.5	38	4.0	3,410	24	Yes	None
	3*	3	76	23	205	L	Ext.	Conc.	3.94	4.39	0.80	1.5	38	4.0	3,500	24	No	Bush Hammer
	3*	3	76	23	205	S	Ext.	Conc.	3.94	4.39	0.80	1.5	38	4.0	3,500	24	No	None
	4@	2	51	23	205	L	Ext.	Epoxy							10,750	75	No	Sand Blast Primer
	4@	2	51	23	205	S	Flush	Epoxy							10,750	75	No	Sand Blast Primer
	5	2	51	23	205	L	Ext.	Conc.	3.96	4.42	0.80	1.5	38	3.5			No	Epoxy Adhesive
	5	2	51	23	205	S	Flush	Conc.	3.96	4.42	0.80	1.5	38	3.5			No	Epoxy Adhesive
	6*	1	25	23	205	L	Flush	Mortar	10.90	12.16	0.44				10,400	72	No	Sand Blast
	6*	1	25	23	205	S	Ext.	Epoxy							10,400	72	Yes	Sand Blast Primer
Stressteel - 22 mm diam. (B)	7	0	0	26	231	L	Ext.	Conc.	3.90	4.35	0.80	1.75	44	5.0	3,450	24	No	None
	7	0	0	26	231	S	Flush	Conc.	3.90	4.35	0.80	1.75	44	5.0	3,450	24	No	None
	8	2	51	26	231	L	Ext.	Conc.	3.90	4.35	0.80	1.5	38	4.0	3,500	24	Yes	None
	8	2	51	26	231	S	Ext.	Conc.	3.90	4.35	0.80	1.5	38	4.0	3,500	24	Yes	Bush Hammer
	9*	3	76	26	231	L	Ext.	Conc.	3.90	4.35	0.80	2	51	5.0	2,510	17	No	Bush Hammer
	9*	3	76	26	231	S	Ext.	Conc.	3.90	4.35	0.80	2	51	5.0	2,510	17	No	Retarding Agent
	10	3	76	26	231	L	Flush	Conc.	3.92	4.37	0.80	1.5	38	4.3			No	Epoxy Adhesive
	10	3	76	26	231	S	Ext.	Conc.	3.92	4.37	0.80	1.5	38	4.3			No	Epoxy Adhesive
	11*	1	25	26	231	L	Flush	Epoxy							10,930	76	No	Sand Blast Primer
	11*	1	25	26	231	S	Ext.	Epoxy							10,930	76	No	Sand Blast Primer
	12	1	25	26	231	L	Ext.	Epoxy							7,800	54	Yes	Sand Blast Primer
	12	1	25	26	231	S	Flush	Mortar	10.90	12.16	0.44				9,320	65	No	Sand Blast
Prescon 8 - 6.4 mm diam.	13*+	0	0	30	267	L	Ext.	Conc.	3.90	4.35	0.80	1.75	44	5.0	3,090	22	No	None
	13*+	0	0	30	267	S	Ext.	Conc.	3.91	4.36	0.80	1.75	44	4.6	3,570	25	No	Bush Hammer
	14	1	25	30	267	L	Ext.	Conc.	3.93	4.38	0.80	1.5	38	4.2	3,430	24	Yes	None
	14	1	25	30	267	S	Ext.	Conc.	3.93	4.38	0.80	1.5	38	4.2	3,430	24	Yes	Bush Hammer
	15*	3	76	30	267	L	Ext.	Conc.	3.94	4.39	0.80	2	51	4.0			No	Retarding Agent
	15*	3	76	30	267	S	Ext.	Conc.	3.94	4.39	0.80	2	51	4.0			No	Epoxy Adhesive
	16	2	51	30	267	L	Ext.	Epoxy							11,320	79	No	Sand Blast Primer
	16	2	51	30	267	S	Ext.	Epoxy							11,320	79	Yes	Sand Blast Primer
Ryerson 12 - 6.4 mm diam.	17@	3	76	42	374	L	Ext.	Conc.	3.90	4.35	0.80	1.5	38	5.0	3,480	24	No	None
	17@	3	76	42	374	S	Ext.	Conc.	3.90	4.35	0.80	1.5	38	5.0	3,480	24	No	Bush Hammer
	18	0	0	42	374	L	Ext.	Conc.	3.94	4.39	0.80	2	51	4.0			Yes	Bush Hammer
	18	0	0	42	374	S	Ext.	Conc.	3.94	4.39	0.80	2	51	4.0			Yes	None
	19*	2	51	42	374	L	Ext.	Conc.	3.90	4.35	0.80	1.25	32	5.0	3,340	23	No	Retarding Agent
	19*	2	51	42	374	S	Ext.	Conc.	3.90	4.35	0.80	1.25	32	5.0	3,340	23	No	Sand Blast Epoxy Adhesive
	20@	1	25	42	374	L	Ext.	Epoxy							9,870	69	Yes	Sand Blast Primer
	20@	1	25	42	374	S	Ext.	Epoxy							9,870	69	No	Sand Blast Primer

* Autopsied 1973 - 1974
@ Autopsied 1983
\+ Unbonded Tendon

DETAILS OF END ANCHORAGE PROTECTION
TABLE 1

P.T. System	Beam No.	Land or Sea End	Anchor Protection Surface Preparation	Year Inspected											
				1963(3)	1968(3)	1972(4)	1973	1974	1978	1980	1982	1983	1988	1990	1994
				Number of Winters											
				2	7	11	12	13	17	19	21	22	27	29	33
				Number of Freezing-Thawing Cycles											
				195	917	1441			2208	2441	2676	2764	3353	3594	3881+
FLUSH CONCRETE ANCHORAGE PROTECTION															
Fressy	5	S	Sand Blast then Epoxy Adhesive			Excellent			Excellent		Excellent		Excellent	Excellent	Excellent
Stress	10	L	Sand Blast then Epoxy Adhesive			Excellent			Excellent		Good				Excellent
Fressy	1	L	None			Excellent		Beam Autopsied							
Stress	7	S	None			Good - Crack Around Pocket			Failed		Failed-Concrete Lost Around Pocket				Failed Anchor Holding
FLUSH EPOXY ANCHORAGE PROTECTION															
Fressy	4	S	Sand Blast then Primer			Failed - Pocket Crack			Failed	Failed	Failed	Beam Autopsied			
Stress	11	L	Sand Blast then Primer			Excellent		Beam Autopsied							
FLUSH MORTAR ANCHORAGE PROTECTION															
Fressy	6	L	Sand Blast			Excellent		Beam Autopsied							
Stress	12	S	Sand Blast			Excellent			Excellent		Good		Good	Good	Failed

Typical Pocket - Flush Anchorage Protection

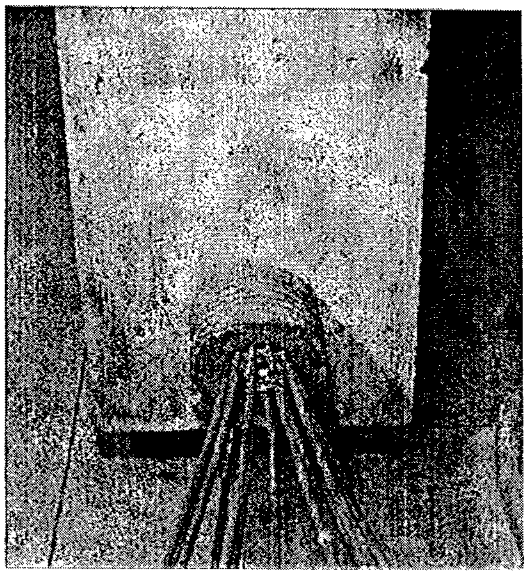
END ANCHORAGE PROTECTION GROUPED BY TYPES WITH OBSERVED CONDITIONS
FROM 1963 TO 1994

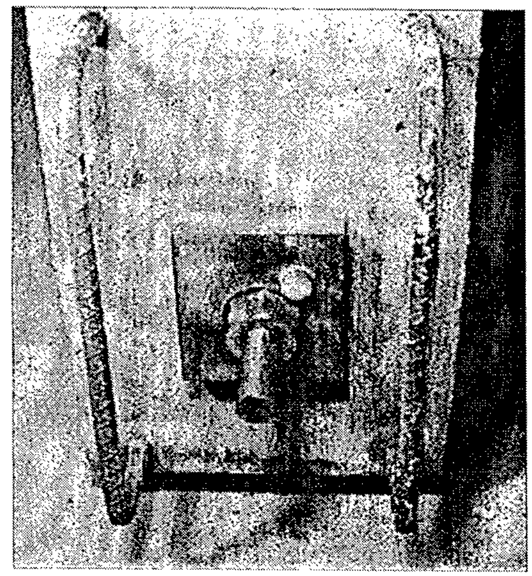
EXTERIOR ANCHORAGE PROTECTION
TABLE 2.1

P.T. System	Beam No.	Land or Sea End	Anchor Protection Surface Preperation	Year Inspected											
				1963(3)	1968(3)	1972(4)	1973	1974	1978	1980	1982	1983	1988	1990	1994
				Number of Winters											
				2	7	11	12	13	17	19	21	22	27	29	33
				Number of Freezing-Thawing Cycles											
				195	917	1441			2208	2441	2676	2764	3353	3594	3881+
EXTERIOR CONCRETE ANCHORAGE PROTECTION WITH NO REINFORCEMENT															
Fressy	1	S	Retarding Agent			Good		Beam Autopsied							
Stress	9	S	Retarding Agent			Failed	Beam Autopsied								
Press	15	L	Retarding Agent			Very Good	Beam Autopsied								
Ryer	19	L	Retarding Agent	End Protection Loose		Failed	Beam Autopsied								
Fressy	5	L	Sand Blast then Epoxy Adhesive			Poor			Failed-Concrete Deteriorated		Failed		Failed	Failed	Failed-Anchor Corrosion but holding
Stress	10	S	Sand Blast then Epoxy Adhesive			Failed-debonded Concrete Deterioration			Failed-End Protection Off		Failed		Failed	Failed	Failed
Press	15	S	Sand Blast then Epoxy Adhesive			Failed	Beam Autopsied								
Ryer	19	S	Sand Blast then Epoxy Adhesive	End Protection Off		Failed	Beam Autopsied								
Stress	9	L	Bush Hammer			Failed - End Protection Off	Beam Autopsied								
Press*	13	S	Bush Hammer			Good	Beam Autopsied								
Ryer	17	S	Bush Hammer	End Protection Off	End Cover Completely	Failed-End Protection Off			Failed		Failed	Beam Autopsied			
Fressy	3	L	Bush Hammer			Very Good	Beam Autopsied								
Fressy	3	S	None			Very Good	Beam Autopsied								
Stress	7	L	None	Bond Line Crack	Failed-End Protection Off						Failed		Failed	Failed	Failed Anchor Holding
Press*	13	L	None			Poor	Beam Autopsied								
Ryer	17	L	None	End Protection Loose		Failed-End Protection Off			Failed		Failed	Beam Autopsied			
EXTERIOR CONCRETE ANCHORAGE PROTECTION WITH REINFORCEMENT															
Fressy	2	L	Bush Hammer			Very Good			Very Good	Very Good	Very Good		Very Good	Very Good	Good-Rebar Corrosion vt Tight
Stress	8	S	Bush Hammer	Bond Line Crack		Poor Resteel Caused Spall			Failed				Failed	Failed	Failed
Press	14	S	Bush Hammer	Bond Line Crack		Excellent			Very Good		Good		Good	Good	Good-Rebar Corrosion Joint Tight
Ryer	18	L	Bush Hammer			Good			Failed		Failed			Failed	Failed
Fressy	2	S	None			Very Good			Very Good	Very Good	Very Good			Very Good	Good-Rebar Corrosion vt Tight
Stress	8	L	None			Poor			Poor						Failed-Bond Good
Press	14	L	None	Bond Line Crack		Very Good			Good		Good		Good	Good	Good Rebar Corrosion
Ryer	18	S	None			Good			Failed		Failed		Failed	Failed	Failed

EXTERIOR ANCHORAGE PROTECTION
TABLE 2.2

P.T. System	Beam No.	Land or Sea End	Anchor Protection Surface Preperation	Year Inspected											
				1963(3)	1968(3)	1972(4)	1973	1974	1978	1980	1982	1983	1988	1990	1994
				Number of Winters											
				2	7	11	12	13	17	19	21	22	27	29	33
				Number of Freezing-Thawing Cycles											
				195	917	1441			2208	2441	2676	2764	3353	3594	3881+
EXTERIOR EPOXY ANCHORAGE PROTECTION WITH NO REINFORCEMENT															
Fressy	4	L	Sand Blast then Primer			Good			Failed-Concrete Deteriorated	Failed	Failed	Beam Autopsied			
Stress	11	S	Sand Blast Primer			Good		Beam Autopsied							
Press	16	L	Sand Blast then Primer			Poor-End Block Crack			Failed Concrete Deterioration		Failed		Failed	Failed	Failed-Interface Concrete Deteriorating
Ryer	20	S	Sand Blast then Primer			Failed-Concrete Deterioration			Failed-45° Crack		Failed	Beam Autopsied			
EXTERIOR EPOXY ANCHORAGE PROTECTION WITH REINFORCEMENT															
Fressy	6	S	Sand Blast Primer			Good		Beam Autopsied							
Stress	12	L	Sand Blast then Primer			Very Good			Failed -Concrete Deterioration		Failed		Failed	Failed	Failed
Press	16	S	Sand Blast then Primer			Very Good			Poor		Failed		Failed	Failed	Failed-Interface Concrete Deteriorating
Ryer	20	L	Sand Blast then Primer			Very Good			Failed		Failed	Beam Autopsied			





Typical Exterior Anchorage Protection

No Reinforcement **With Reinforcement**

EXTERIOR ANCHORAGE PROTECTION
TABLE 2.3

Mixtures Used to Fabricate Post-Tensioned Beams

A. Beams Proper (excluding the grout and anchorage protection)

Cement	Max Size Aggregate in.	Air Content %	Water-Cement Ratio (by Wt)*	Slump in.	Cement Factor bags*/cu yd	Nominal Compressive Strength psi (28 Days Age)
Type III (high-early-strength)	3/4	4.0-5.0	0.52 (5.85 gal/bag)	1-1/2 to 2	5.98-6.05	6000

B. Anchorage Protection (excluding epoxy mixture)

Cement	Max Size Aggregate in.	Air Content %	Water-Cement Ratio (by Wt)*	Slump in.	Cement Factor bags*/cu yd	Nominal Compressive Strength psi (28 Days Age)
Type III (high-early-strength)	3/4	3.5-5.0	0.80 (9.03 gal/bag)	1-1/4 to 2	3.90-3.96	3000

C. Epoxy Concrete Protection

Cement	Max Size Aggregate, in.	Mixture Proportions (by Wt), Epoxy Binder:Sand:Coarse Aggregate	Compressive Strength, psi (28 Days Age)
None	3/4	2.83:7.00:10.00	9,320-11,320

D. Mortar Mixtures

Cement	Max Size Aggregate, in.	Water-Cement Ratio (by Wt)*	Cement Factor bags*/cu yd	Compressive Strength, psi (28 Days Age)
Type III (high-early-strength)	100% passing No. 4 sieve	0.44 (4.95 gal/bag)	10.90	7710-7800

E. Grout Mixtures

Cement	Water Cement Ratio (by Wt)*	Compressive Strength, psi (7 Days Age)	Linear Expansion, % (3 Days Age)
Type III (high-early-strength)	0.40-0.49 (4.51-5.53 gal/bag)	3740-6430	0-7

Note: All grouts were neat cement grouts except that used for beam 14, which was a natural sand grout (100 percent passing No. 30 sieve). All of the grouts contained a small amount of aluminum powder (1 to 3 g per bag of cement).

* One Bag = 94 lb. of cement = 45 kg (mass)

TABLE 3

WES DESIGNATION	A	B	C	D
Post-Tensioning System	**Freyssinet**	**Stressteel**	**Prescon**	**Ryerson**
Steel Diameter - Inches	0.196 (5 mm)	0.875 (22 mm)	0.25 (6 mm)	0.25 (6 mm)
No. of Wires or Bars	12	1	8	12
Ultimate Strength - Ksi	250 (1,724 MPa)	145 (1000 MPa)	240 (1655 MPa)	240 (1655 MPa)
Anchorage Type	Concrete Cone Wedge	Tapered Thread	Button Headed Wire	Button Headed Wire
Initital Force-Tons	42 (38 Mg)	35 (32 Mg)	35 (32 Mg)	50 (45 Mg)
Estimated Final Force -Tons	23 (21 Mg)	26 (24 Mg)	30 (27 Mg)	42 (38 Mg)
Sheath* O.D. Inches	1-5/8 (41 mm)	1.25 (32 mm)	1.25 (32 mm)	1-5/8 (41 mm)

* All sheaths bright flexible metal except Beam 13, which was paper wrapped unbonded.

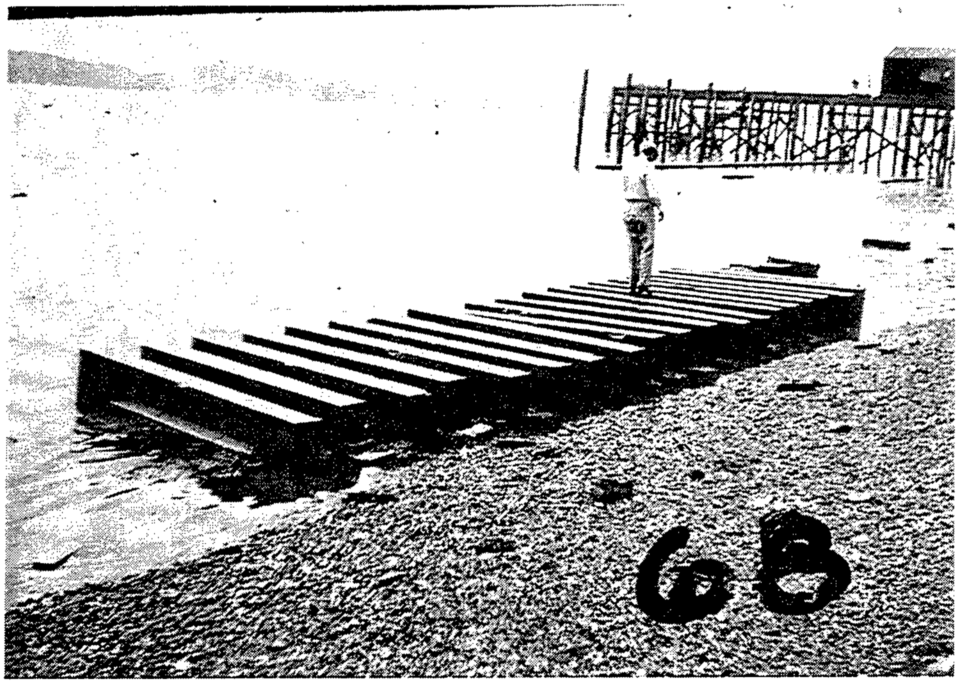
POST - TENSIONING SYSTEMS USED

TABLE 4

P.T. System	Beam No.	Land or Sea End	End Protection Surface Preparation	Date of Failure	Date of Autopsy	Condition at Autopsy	Condition as of 1994
FLUSH ANCHOR PROTECTION							
Concrete Anchor Protection							
Fressy	***5***	***S***	***Sand Blast Epoxy Adhesive.***				***Excellent***
Stress	***10***	***L***	***Sand Blast Epoxy Adhesive.***				***Excellent***
Fressy	1	L	None		1974	Excellent	
Stress	7	S	None	1978			Failed
Epoxy Anchor Protection							
Fressy	4	S	Sand Blast Primer	1972	1983	Failed	
Stress	11	L	Sand Blast Primer		1974	Excellent	
Mortar Anchor Protection							
Fressy	6	L	Sand Blast		1974	Excellent	
Stress	12	S	Sand Blast	1994			Failed
EXTERIOR ANCHOR PROTECTION							
Concrete Anchor Protection with No Reinforcement							
Fressy	1	S	Retarding Agent		1974	Good	
Stress	9	S	Retarding Agent	1972	1973	Failed	
Presc	15	L	Retarding Agent		1973	Very Good	
Ryer	19	L	Retarding Agent	1972	1973	Failed	
Fressy	5	L	Sand Blast Epoxy Adhesive.	1978			Failed
Stress	10	S	Sand Blast Epoxy Adhesive.	1972			Failed
Presc	15	S	Sand Blast Epoxy Adhesive.	1972	1973	Failed	
Ryer	19	S	Sand Blast Epoxy Adhesive.	1963	1973	Failed	
Stress	9	L	Bush Hammer	1972	1973	Failed	
Presc	13	S	Bush Hammer		1973	Good	
Ryer	17	S	Bush Hammer	1963	1983	Failed	
Fressy	3	L	Bush Hammer		1973	Very Good	
Fressy	3	S	None		1973	Very Good	
Stress	7	L	None	1968			Failed
Presc	13	L	None		1973	Poor	
Ryer	17	L	None	1972	1983	Failed	
Concrete Anchor Protection with Reinforcement							
Fressy	**2**	**L**	**Bush Hammer**				**Good**
Stress	8	S	Bush Hammer	1978			Failed
Presc	**14**	**S**	**Bush Hammer**				**Good**
Ryer	18	L	Bush Hammer	1978			Failed
Fressy	**2**	**S**	**None**				**Good**
Stress	8	L	None	1994			Failed
Presc	**14**	**L**	**None**				**Good**
Ryer	18	S	None	1978			Failed
Epoxy Anchor Protection with No Reinforcement							
Fressy	4	L	Sand Blast Primer	1978	1983	Failed	
Stress	11	S	Sand Blast Primer		1974	Good	
Presc	16	L	Sand Blast Primer	1978			Failed
Ryer	20	S	Sand Blast Primer	1972	1983	Failed	
Epoxy with Reinforcement							
Fressy	6	S	Sand Blast Primer		1974	Good	
Stress	12	L	Sand Blast Primer	1978			Failed
Presc	16	S	Sand Blast Primer	1982			Failed
Ryer	20	L	Sand Blast Primer	1978	1983	Failed	

DATE OF END ANCHORAGE PROTECTION FAILURE OR AUTOPSY OR 1994 CONDITION

TABLE 5



1968 - Twenty post tensioned beams partially submerged.

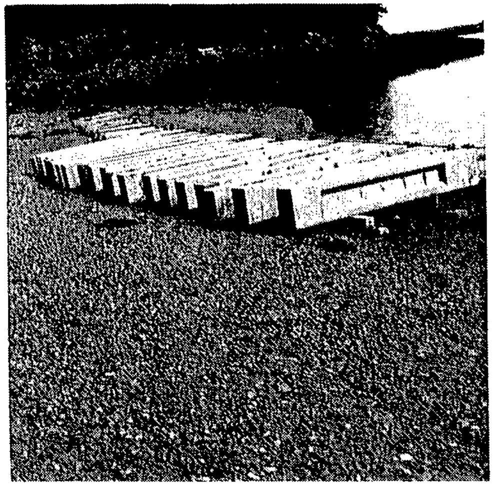
Fig. 1A

1986 - Nine remaining post-tensioned beams in foreground.

Fig 1B

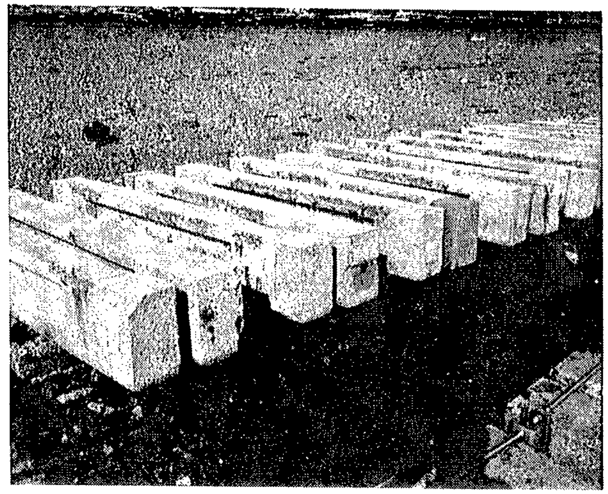
CORPS OF ENGINEERS SEVERE EXPOSURE STATION AT TREAT ISLAND, MAINE

FIG. 1



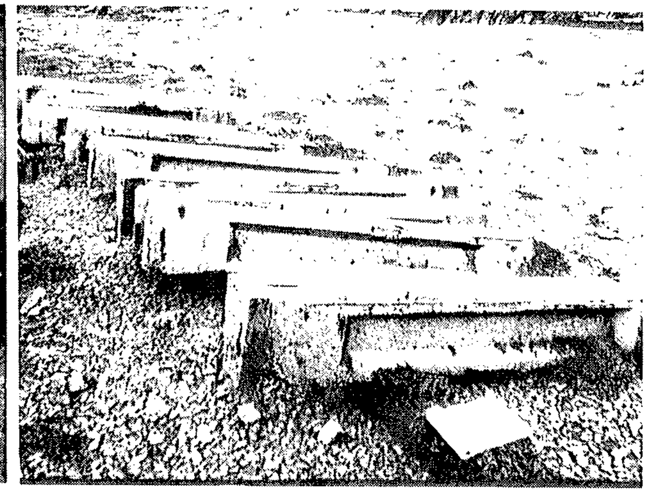
1962 - 20 Beams placed in 1961

Fig. 2A



1972 - 20 beams

Fig. 2B

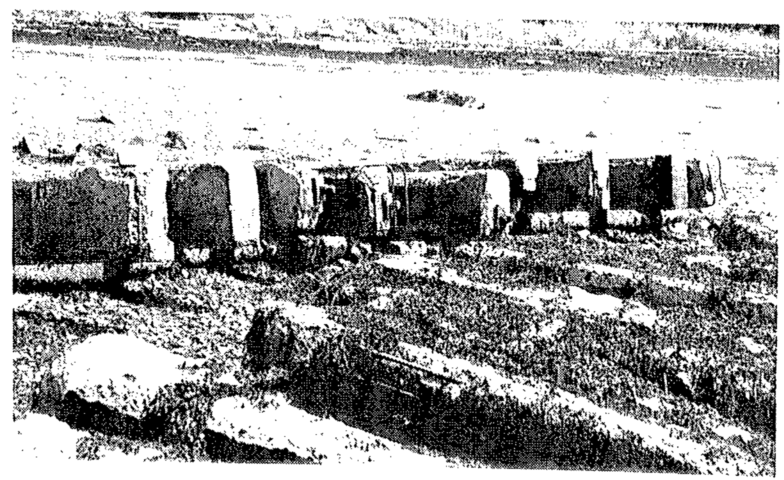


1982 - 12 beams remaining - 8 removed for autopsy at WES in 1973 and 1974

Fig. 2C

1990 - 9 beams remaining - 3 removed for autopsy at WES in 1984

Fig. 2D

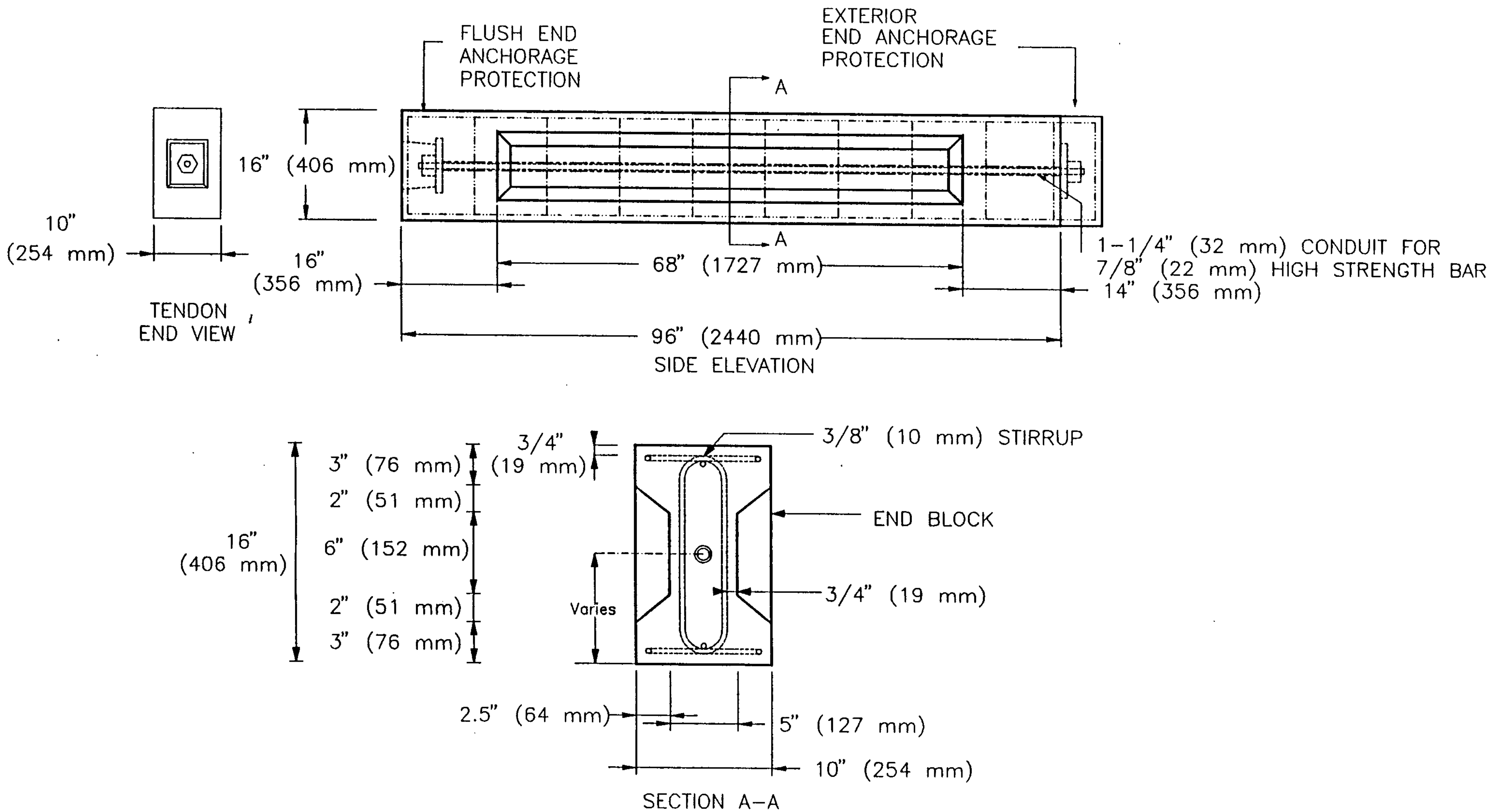


1994 - 9 post-tensioned beams at Treat Island

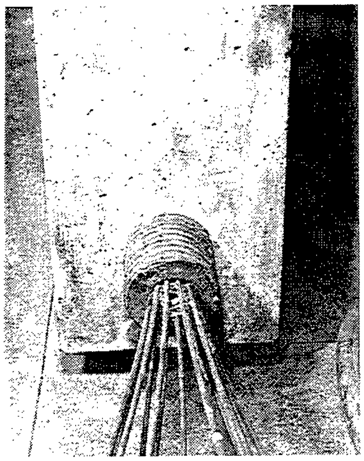
Fig. 2E

OVERVIEW OF THE POST-TENSIONED BEAMS AT TREAT ISLAND IN 1962, 1972, 1982, 1990 AND 1994

FIG. 2

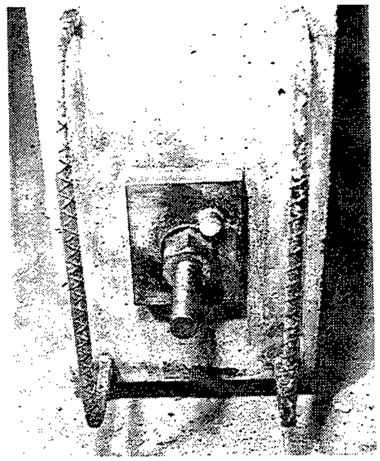


TEST BEAM DETAILS
FIG. 3



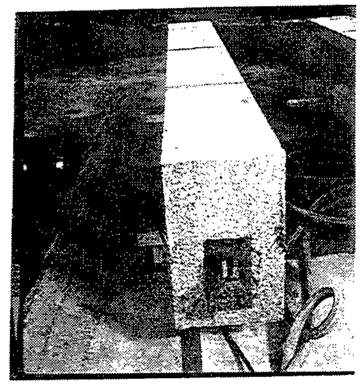
Exterior
No reinforcement
No surface prep.
(Freyssinet tendon
prior to cutting wire)

Fig. 4A



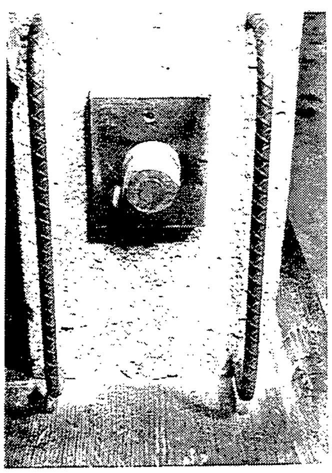
Exterior
With reinforcement
No surface prep.
(High strength bar)

Fig. 4B



Exterior
No reinforcement
Bush hammered
(Prescon tendon)

Fig. 4C



Exterior
With reinforcement
Sandblast prep.
(High strength bar)

Fig. 4D

Exterior
No reinforcement
Bush hammered prep.
(Freyssinet tendon)

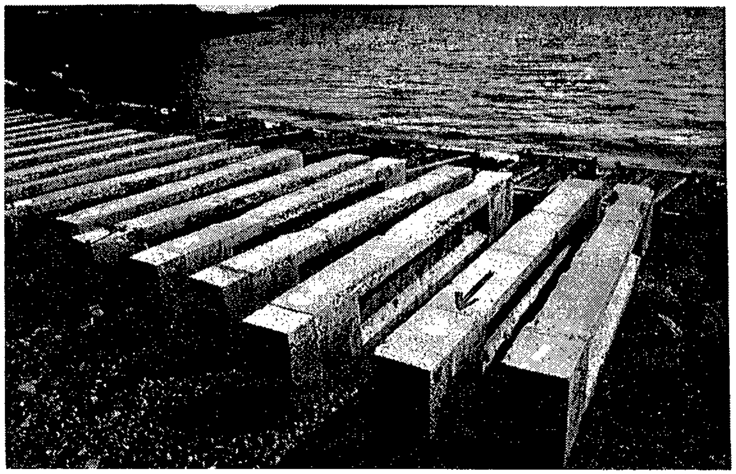
Fig. 4E

Flush
No surface prep.
(High strength bar)

Fig. 4F

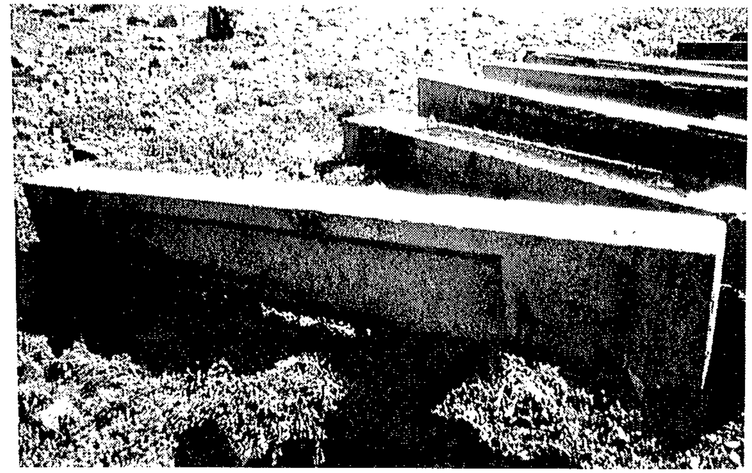
END ANCHORAGE PROTECTION DETAILS PRIOR TO PLACING PROTECTION

FIG. 4



Beam 2 - Looking seaward - 1972

Fig. 5-1A

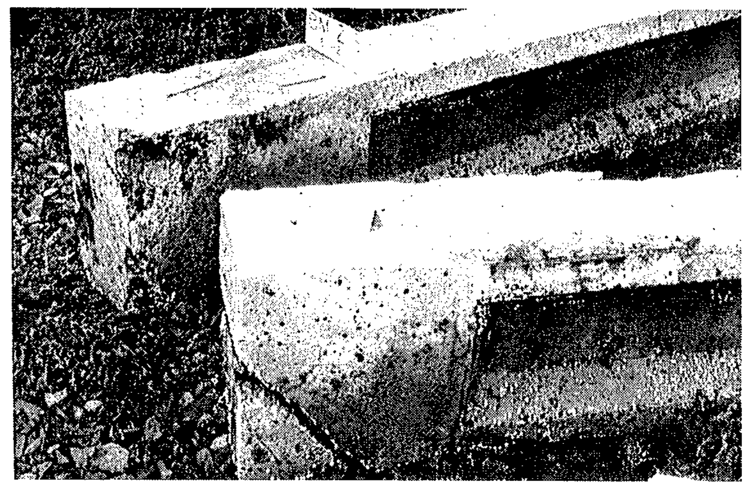


Beam 2 - 1978

Fig. 5-1B

Beam 2 - 1978

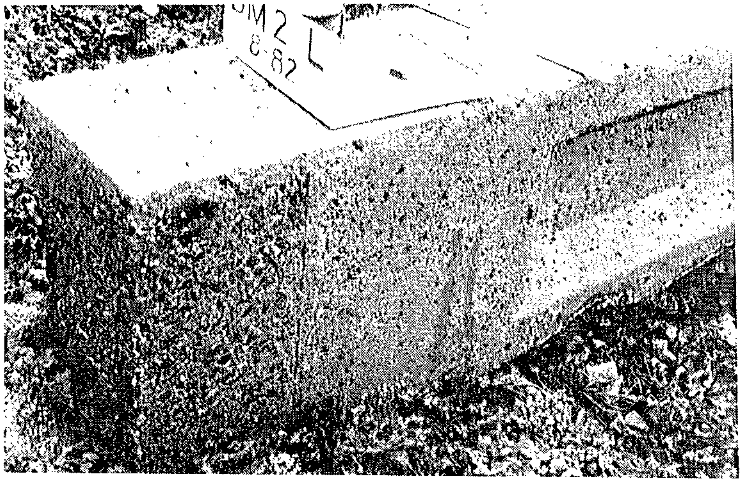
Fig. 5-1C



Beam 2 - Landward end - 1982

Fig. 5-1D

PHOTO HISTORY OF BEAM 2
FIG. 5-1



Beam 2 - Landward end - 1982

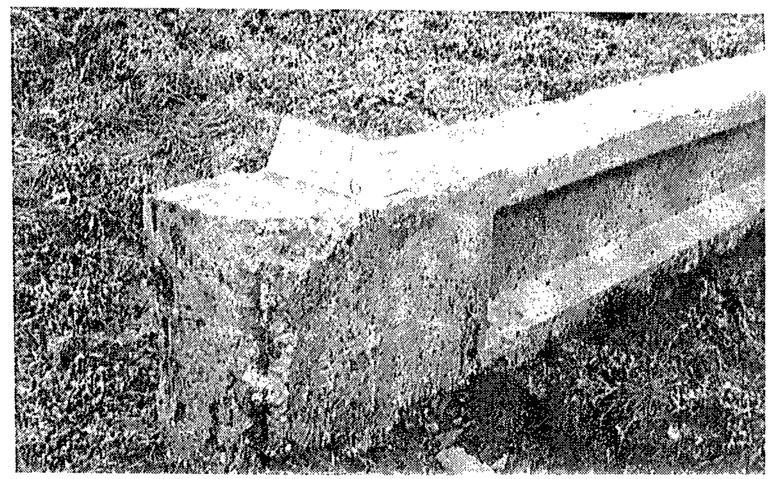
Fig. 5-2E

Beam 2 - Seaward end - 1982

Fig. 5-2F

Beam 2 - Landward end - 1982

Fig. 5-2G

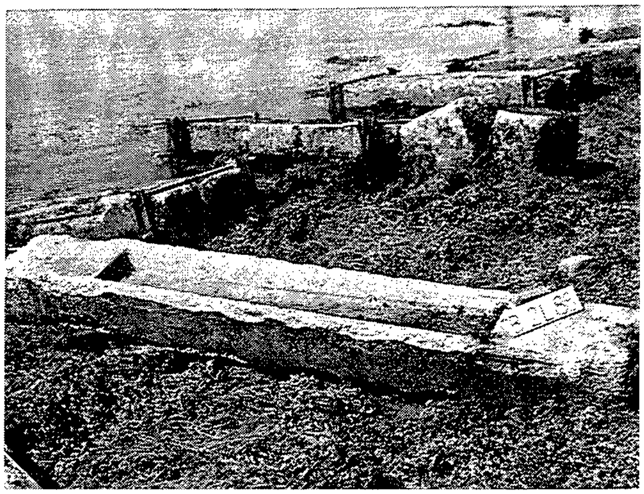


Beam 2 - Landward end - 1986

Fig. 5-2H

PHOTO HISTORY OF BEAM 2 (cont.)

FIG. 5-2

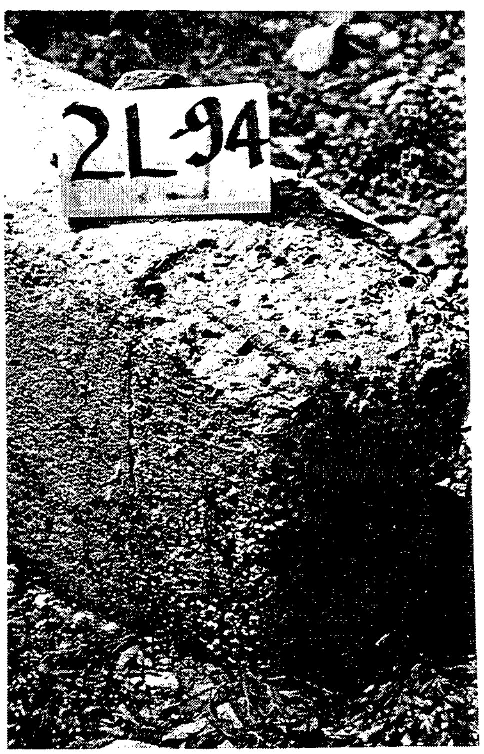


Beam 2 - 1988

Fig. 5-3I

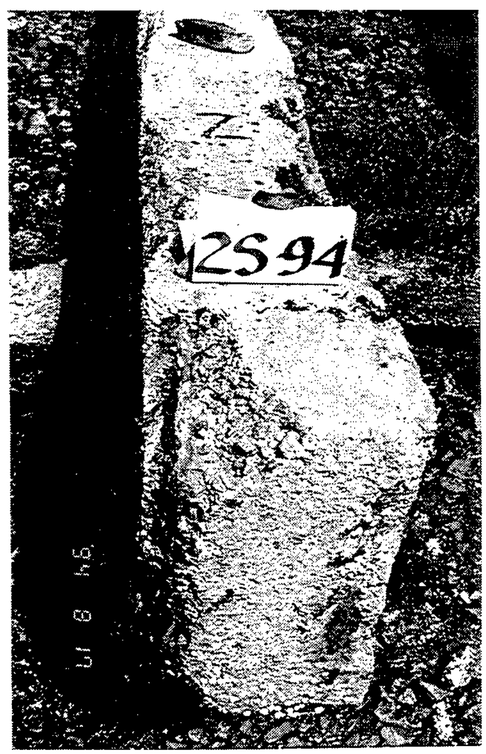
Beam 2 - Seaward end - 1990

Fig. 5-3J



Beam 2 - Landward end - 1994

Fig. 5-3K

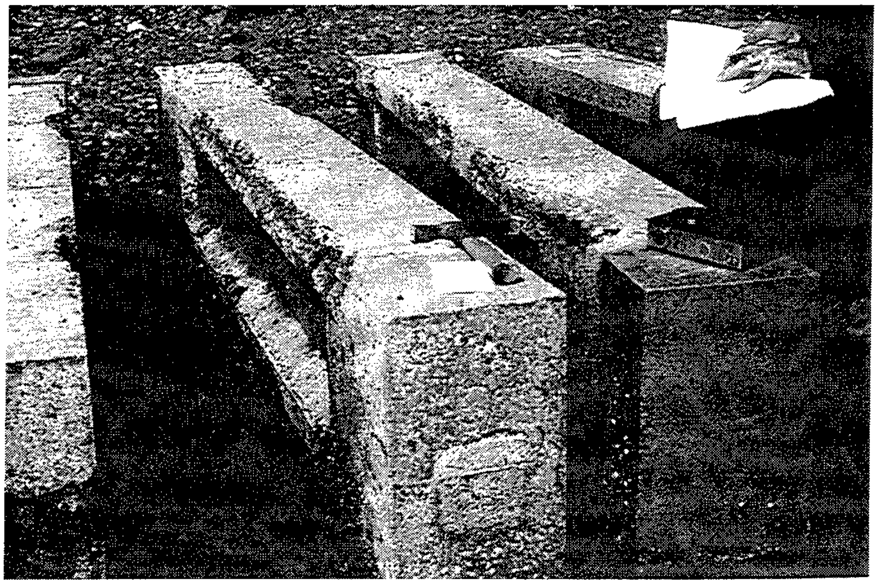


Beam 2 - Seaward end - 1994

Fig. 5-3L

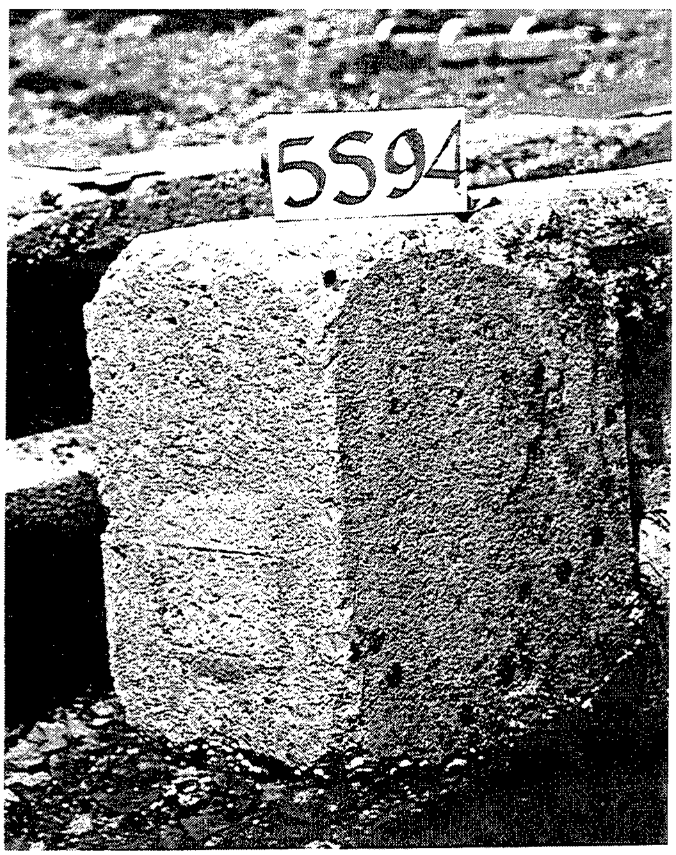
PHOTO HISTORY OF BEAM 2 (cont.)

FIG. 5-3



Beam 5 - Seaward end - 1972

Fig. 6A

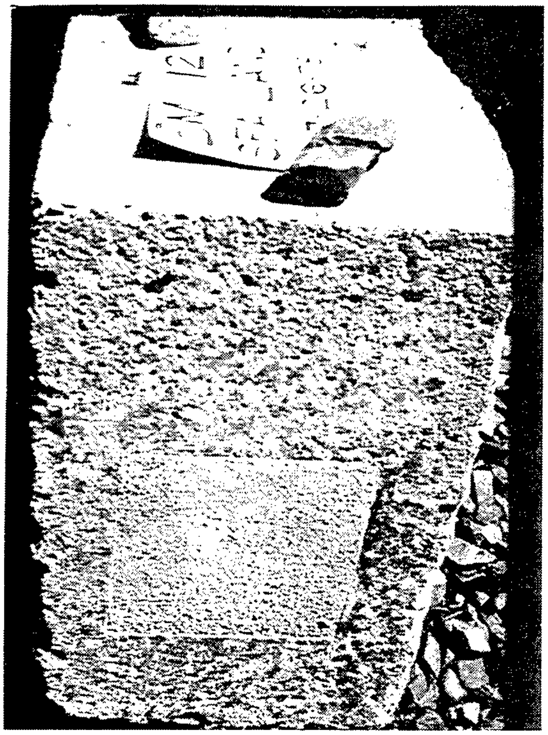


Beam 5 - Seaward end - 1994

Fig. 6B

PHOTO HISTORY OF BEAM 5

FIG. 6



Beam 12 - Seaward end - 1978

Fig. 7-1A

Beam 12 - Seaward end - 1982

Fig. 7-1B

Beam 12 - Seaward end - 1990

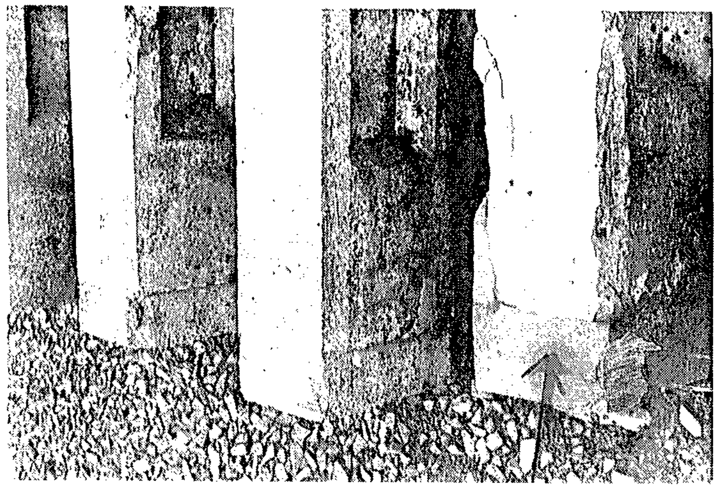
Fig. 7-1C

Beam 12 - Seaward end - 1994

Fig. 7-1D

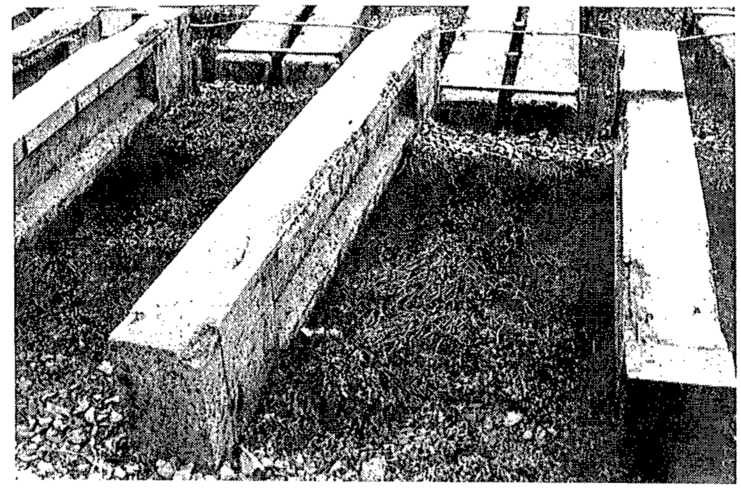
PHOTO HISTORY OF BEAM 12

FIG. 7-1



Beam 12 - Landward end - 1968

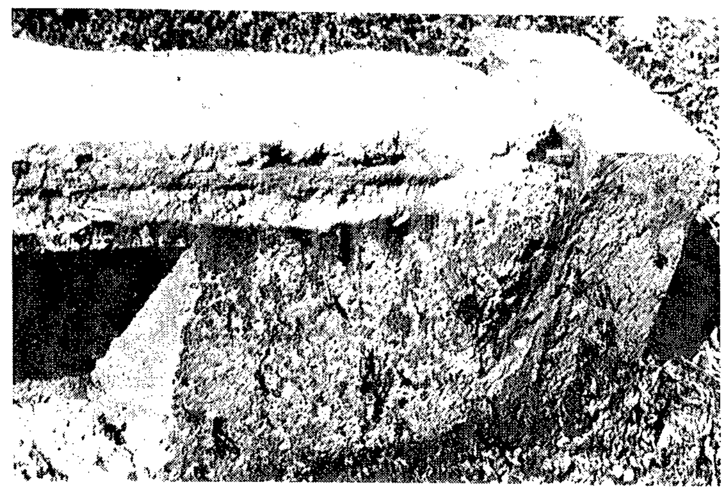
Fig. 7-2E



Beam 12 - Landward end - 1976

Fig. 7-2F

Beam 12- Landward end - 1980
Rebar corrosion into epoxy end protection
Fig. 7-2G



Beam 12 - Landward end - 1980
Edge spalling and rebar corrosion
Fig. 7-2H

PHOTO HISTORY OF BEAM 12 (cont.)

FIG. 7-2

Beam 12 - Landward end - 1984
Corrosion of rebars
Fig. 7-3I

Beam 12 - 1988
Epoxy end cap and rebar corrosion
Fig. 7-3J

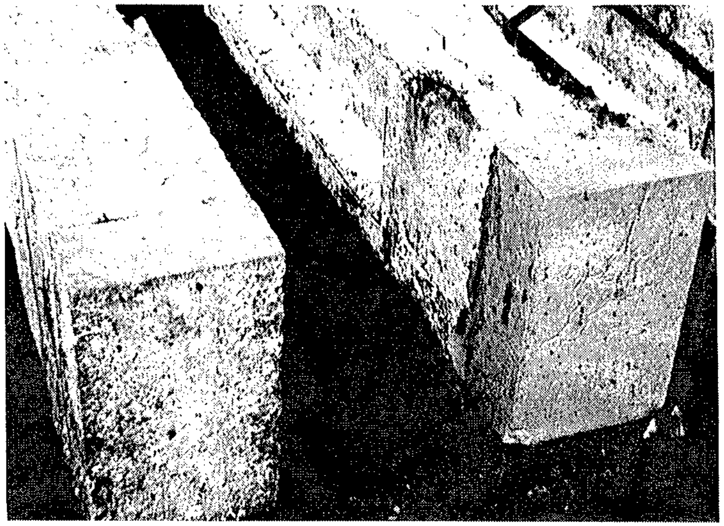
Beam 12 - Landward end - 1994
Concrete and rebar deterioration

Fig. 7-3K

Beam 12 - Landward end - 1994
Concrete and rebar deterioration

Fig. 7-3L

PHOTO HISTORY OF BEAM 12 (cont.)
FIG. 7-3

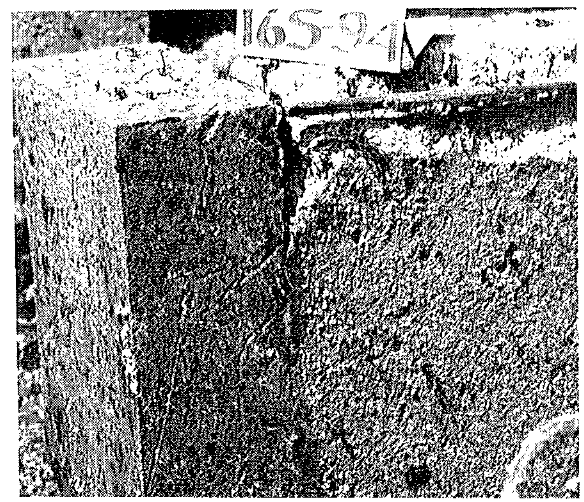


Beam 16 - Seaward end - 1972
Rebar corrosion
Fig. 8-1A

Beam 16 - Seaward end - 1978
Progressive corrosion & spalling
Fig. 8-1B

Beam 16 - Seaward end - 1988

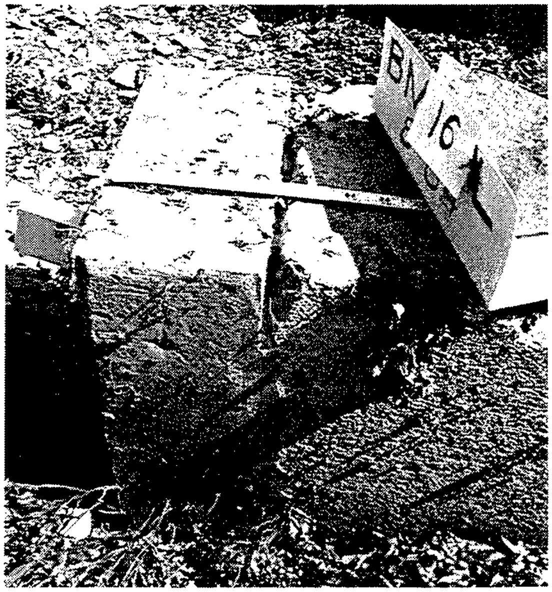
Fig. 8-1C

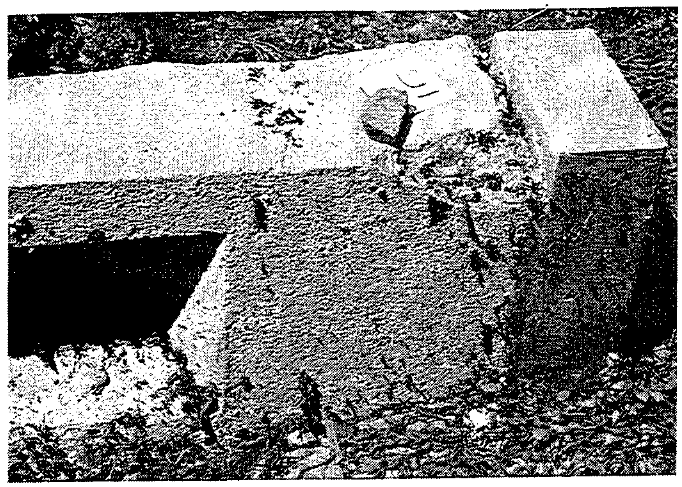


Beam 16 - Seaward end - 1994
Interface failure
Fig. 8-1D

PHOTO HISTORY OF BEAM 16

FIG. 8-1

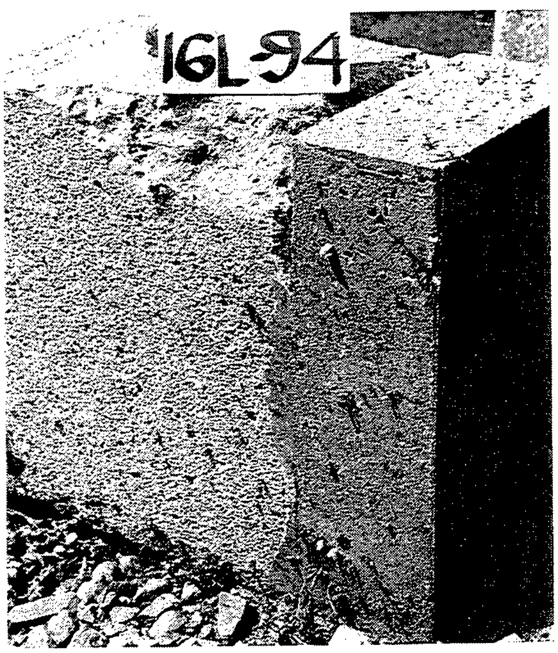




Beam 16 - Landward end - 1984
Concrete interface breakdown
Fig. 8-2E

Beam 16 - Landward end - 1988

Fig. 8-2F



Beam 16 - Landward end - 1994

Fig. 8-2G

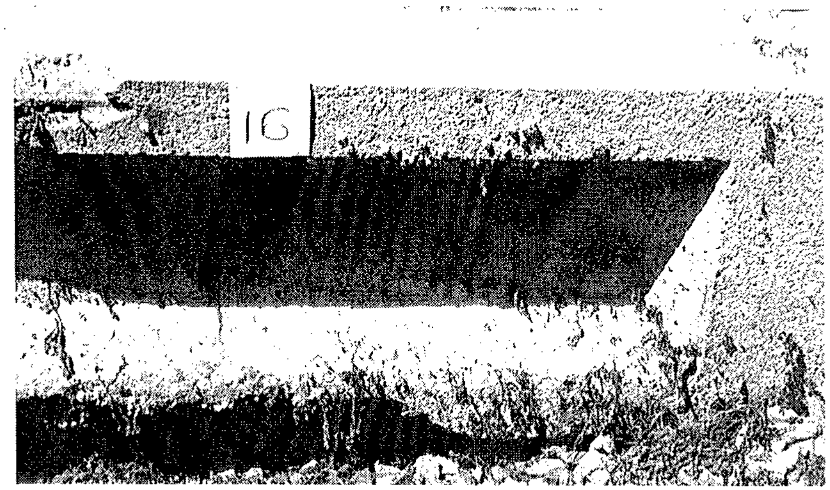
Beam 16 - Landward end - 1994

Fig. 8-2H

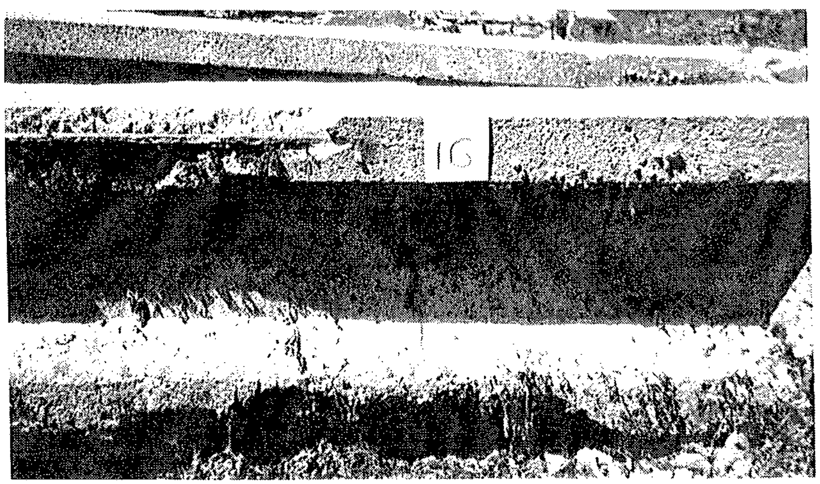
PHOTO HISTORY OF BEAM 16 (cont.)

FIG. 8-2

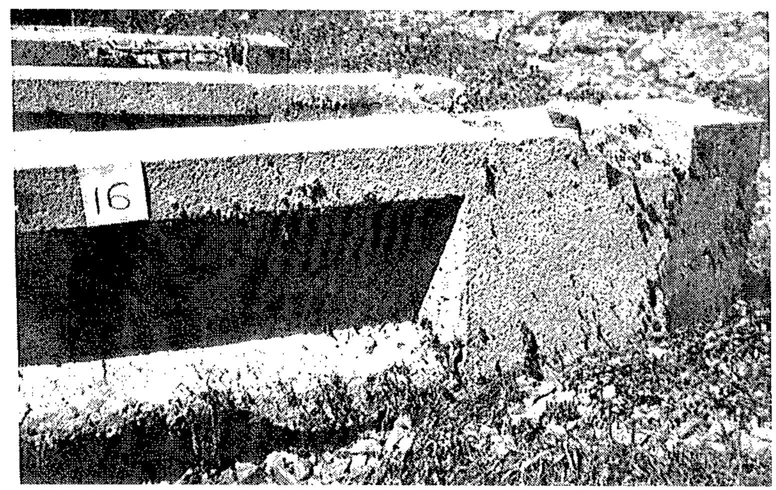
Beam 16 - 1984
First indication of stirrup corrosion
Fig. 8-3I



Beam 16 - 1988
Stirrup corrosion has not progressed
Fig. 8-3J



Beam 16 - 1990
Stirrup corrosion
Fig. 8-3K



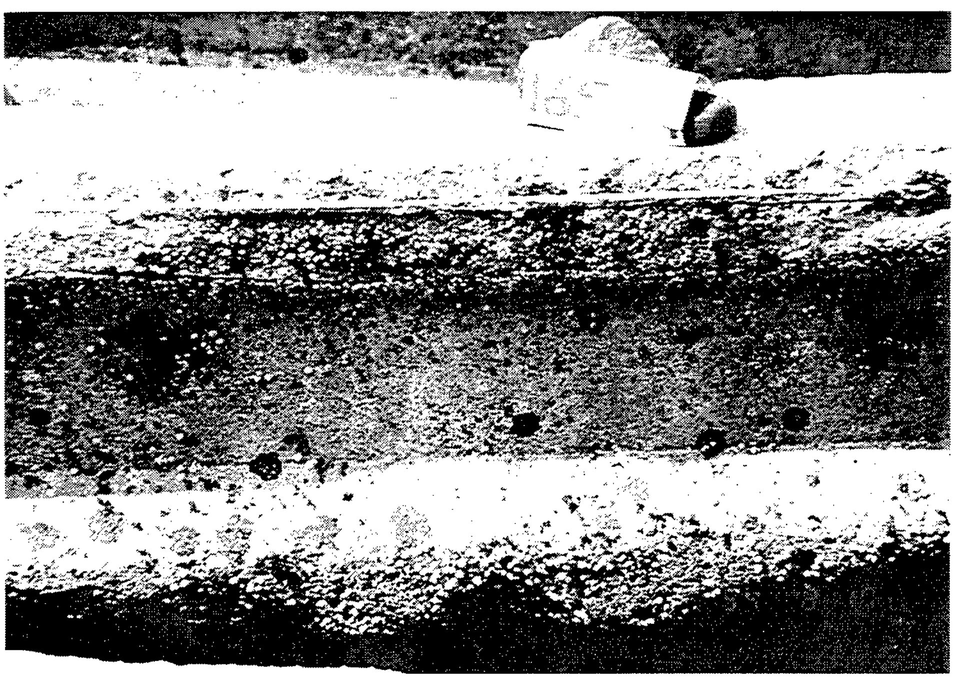
Beam 16 - 1994
Stirrup corrosion
Fig. 8-3L

PHOTO HISTORY OF BEAM 16 (cont.)

FIG. 8-3

Beam 16 - 1990

Fig. 8-4M

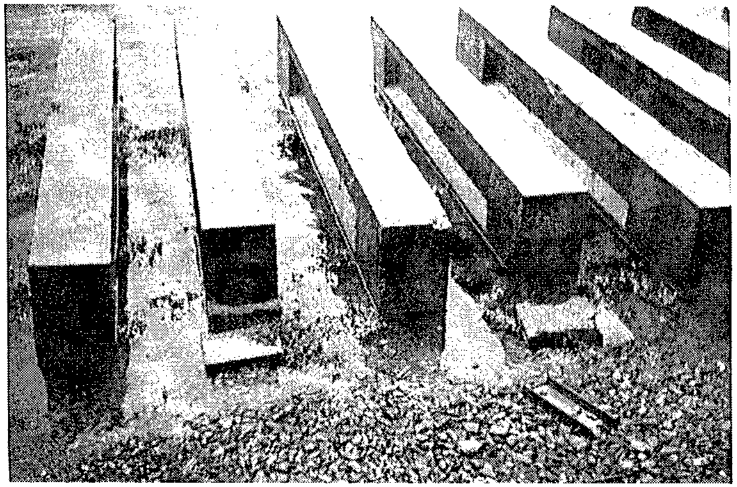


Beam 16 - 1994

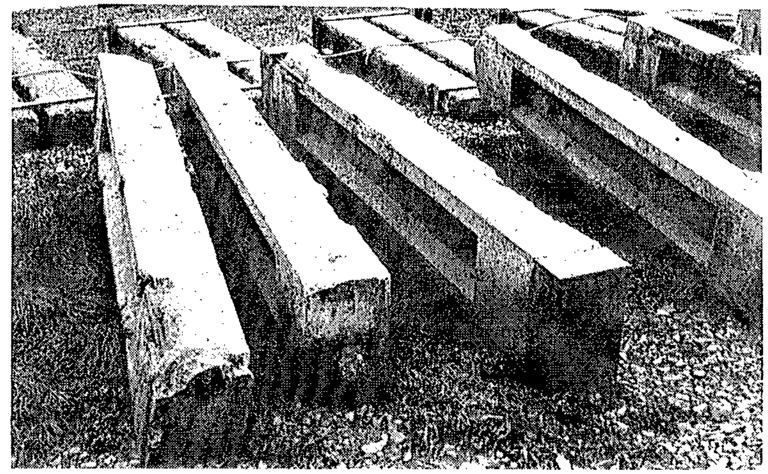
Fig 8-4N

PHOT HISTORY BEAM 16 (cont)

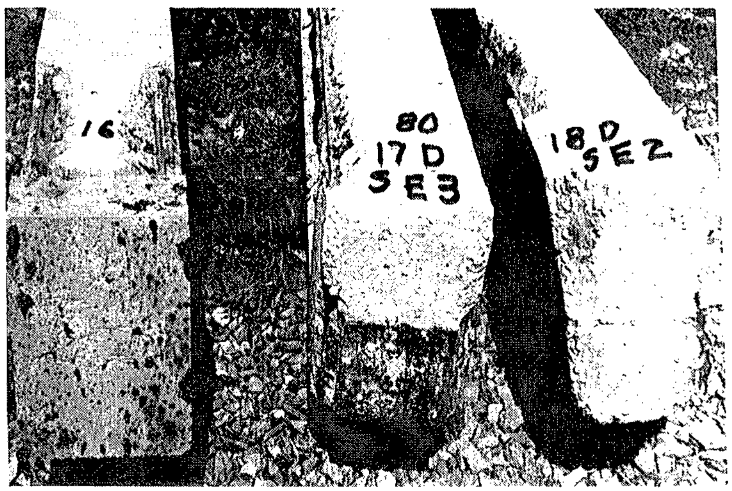
FIG. 8-4



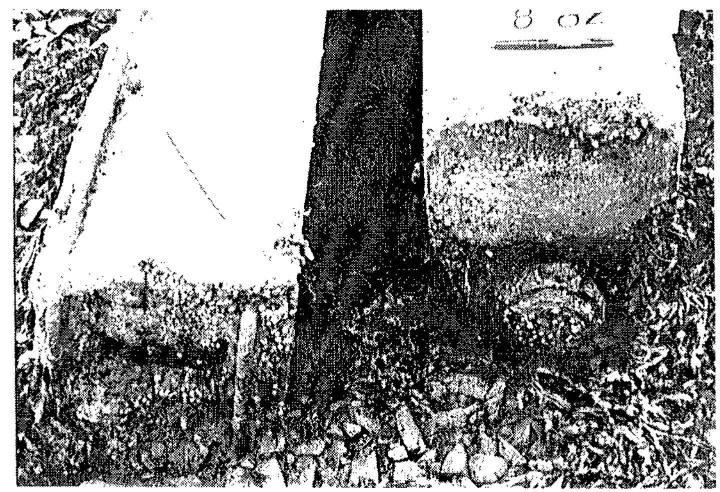
Beam 17 - Landward end - 1972
End protection failed
Fig. 9-1A



Beam 17 - Landward end - 1976
Edge spall
Fig. 9-1B



Beam 17 - Seaward end - 1980
Failed 1963
Fig. 9-1C



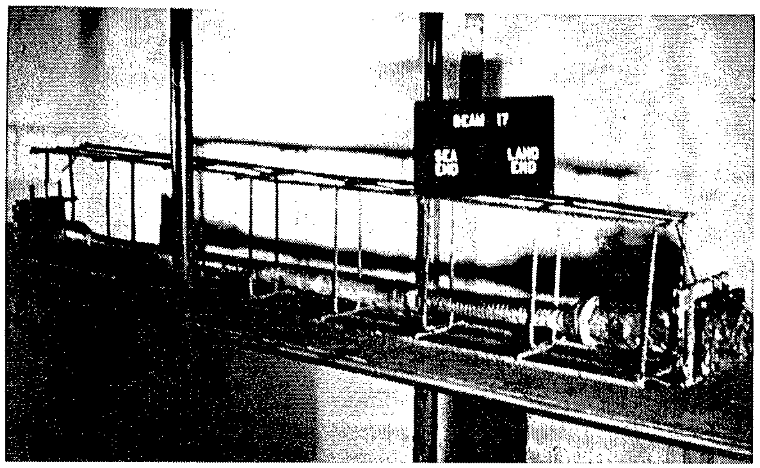
Beam 17 - Landward end - 1982
Before 1984 autopsy
Fig. 9-1D

PHOTO HISTORY OF BEAM 17 - AUTOPSIED 1984

FIG. 9-1

Beam 17 - Landward end - 1984
Prior to autopsy
Fig. 9-2E

Beam 17 - Seaward end - 1984
Prior to autopsy
Fig. 9-2F



Beam 17 - Reinforcement cage - 1984

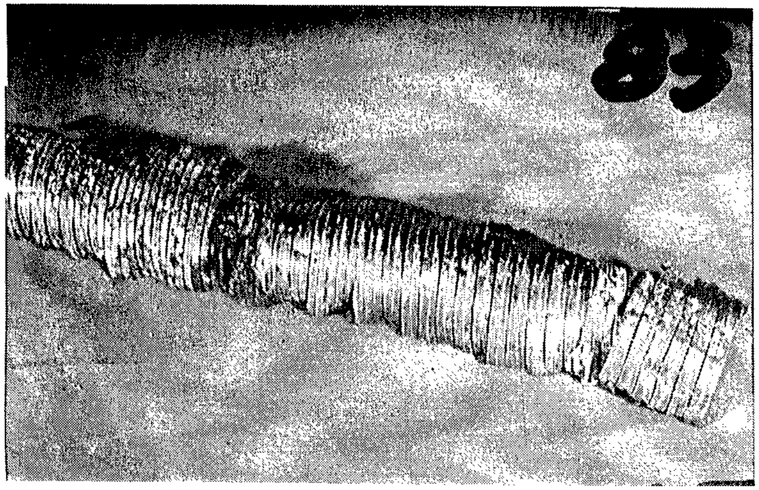
Fig. 9-2G

Beam 17 - Landward end reinforcement, anchorage - 1984

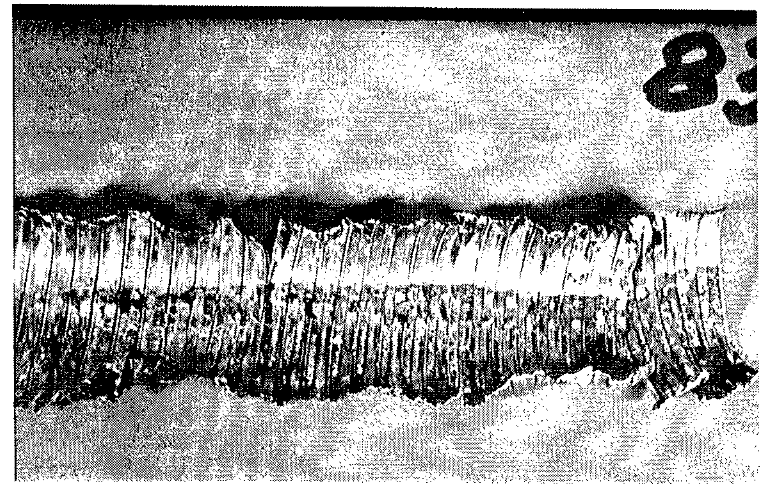
Fig. 9-2H

PHOTO HISTORY OF BEAM 17 - AUTOPSIED 1984 (cont.)

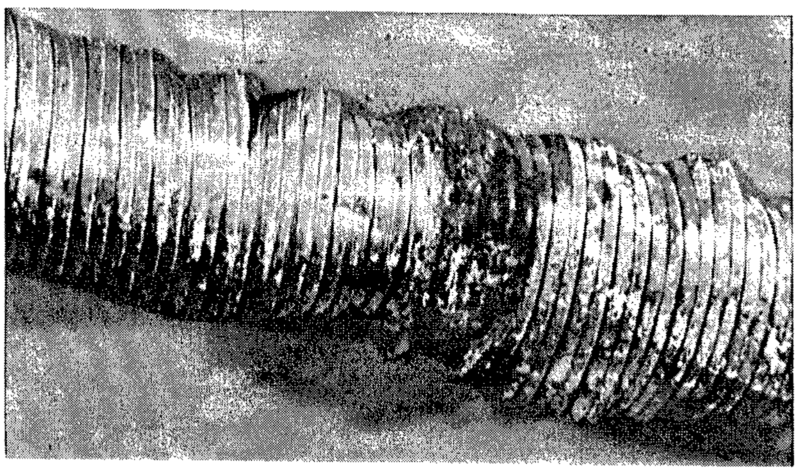
FIG. 9-2



Beam 17 - Outside - 1984
Post-tensioning conduit corrosion
Fig. 9-3I

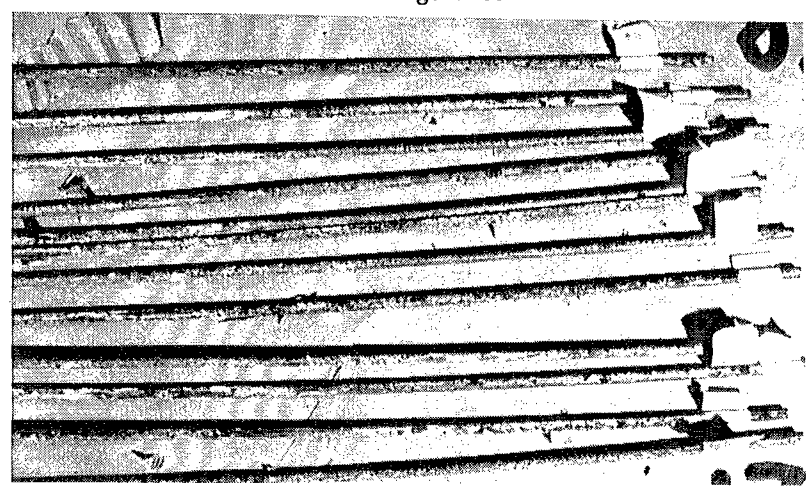


Beam 17 - Inside - 1984
Post-tensioning conduit corrosion
Fig. 9-3J



Beam 17 - Post-tensioning conduit corrosion - 1984

Fig. 9-3K

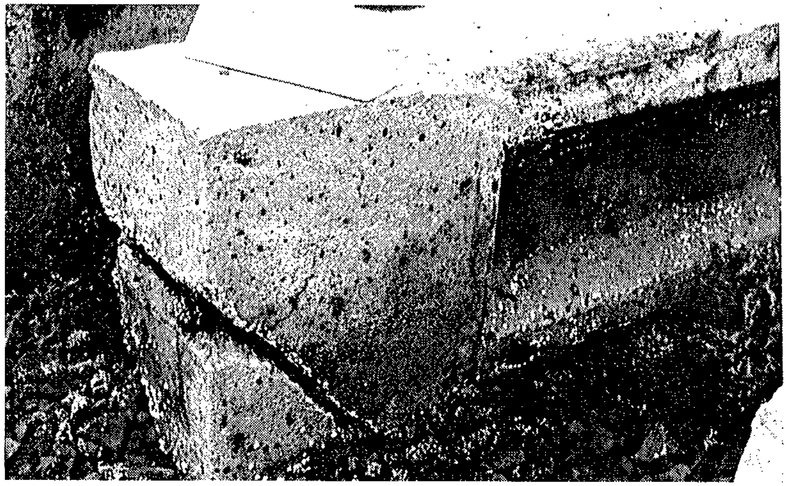


Beam 17 - Prestressing wire corrosion - 1984

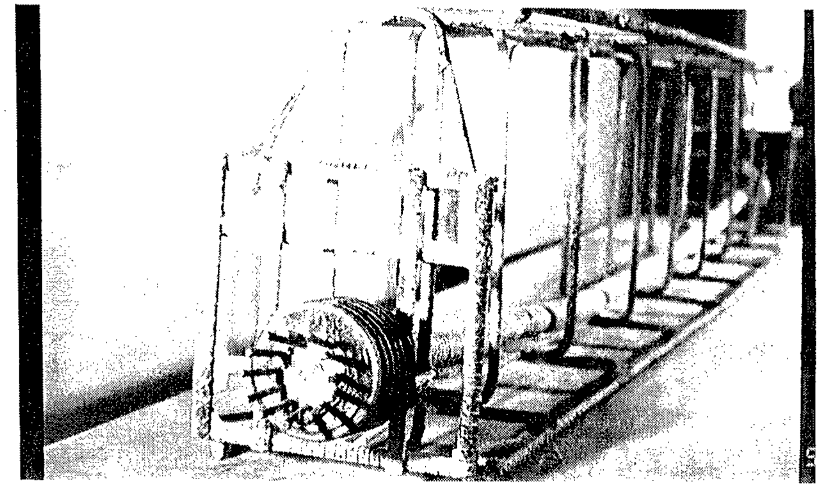
Fig. 9-3L

PHOTO HISTORY OF BEAM 17 - AUTOPSIED 1984 (cont.)

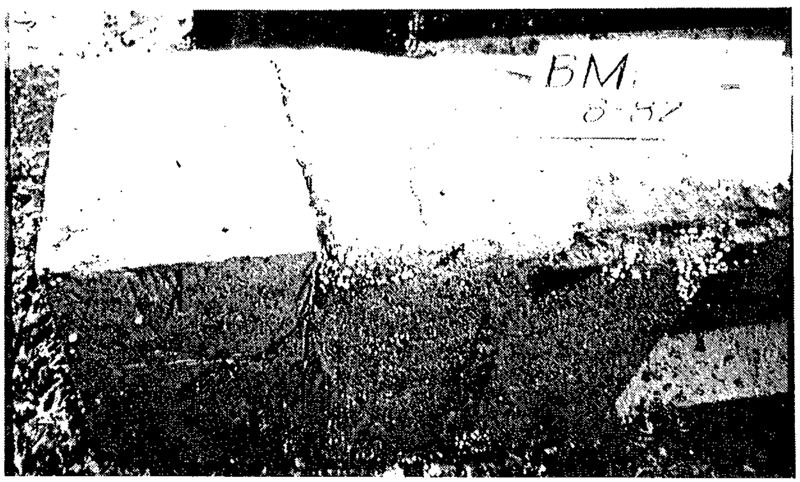
FIG. 9-3



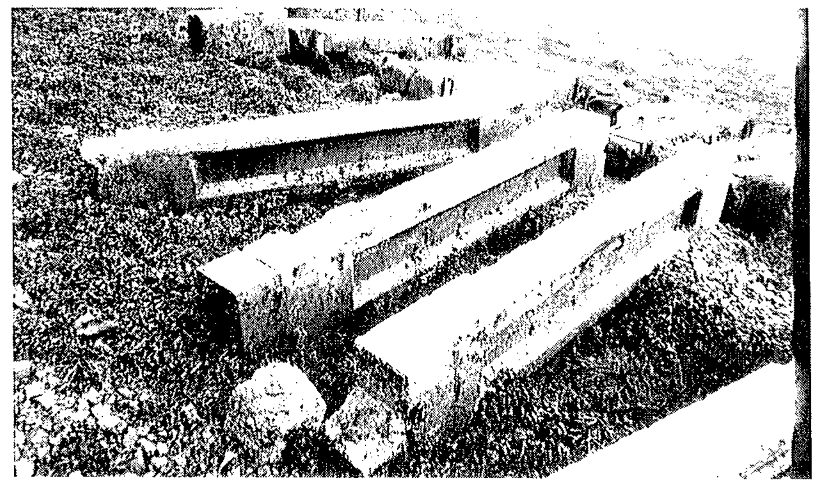
Beam 4 - Seaward end - 1982
Failed flush epoxy anchorage protection
Fig. 10-1A



Beam 10 - Autopsy - 1983
Anchor and reinforcement cage
Fig. 10-1B



Beam 4 - Landward end - 1982
Epoxy external end protection
Fig. 10-1C

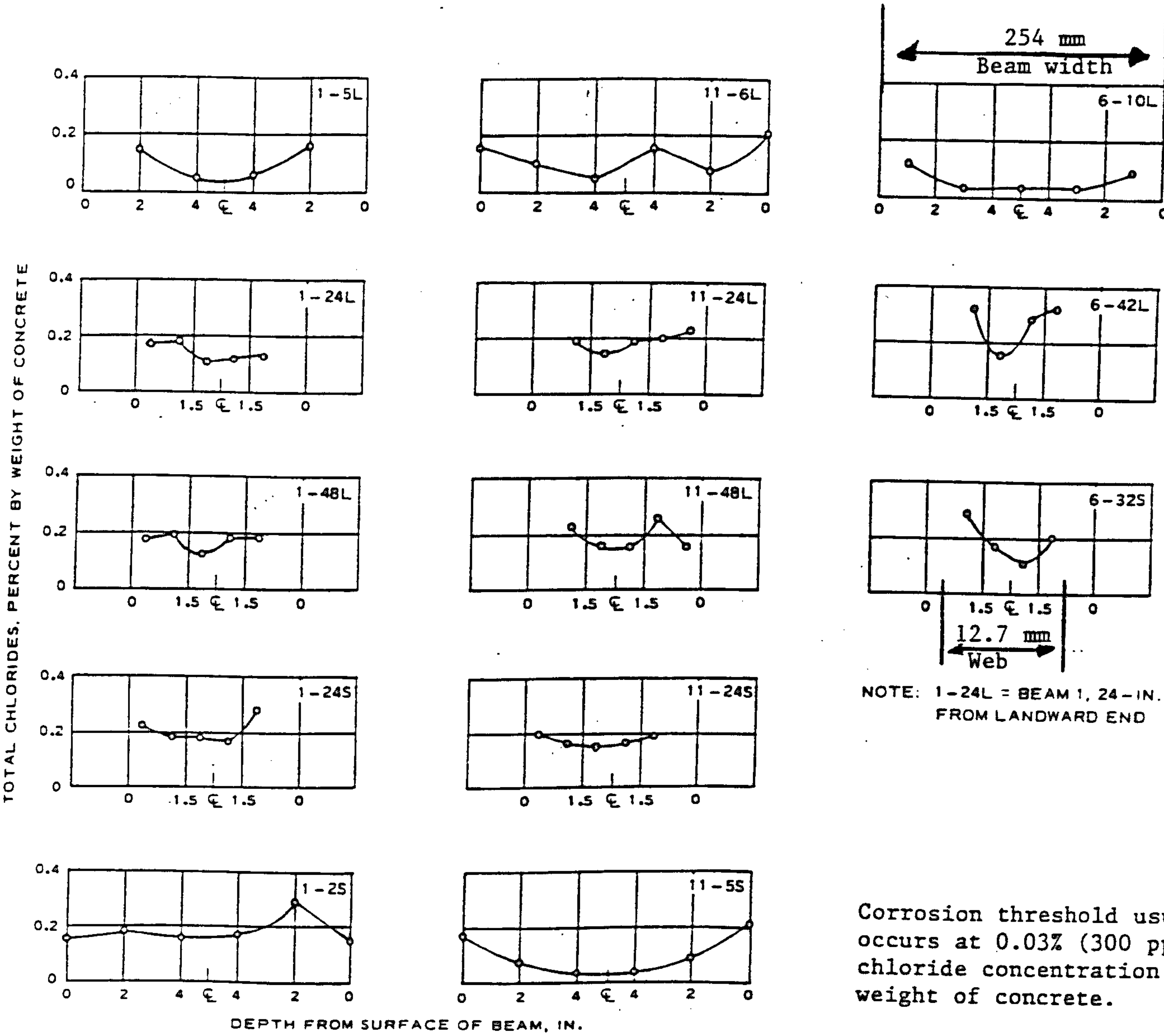


Beam 4 - Landward end - 1982

Fig. 10-1D

EPOXY CONCRETE END ANCHORAGE PROTECTION

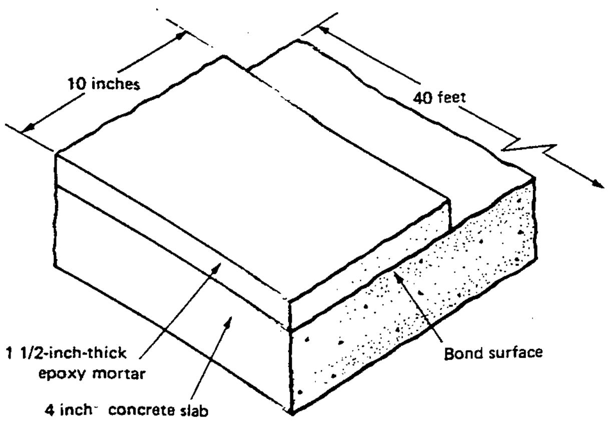
FIG. 10



Corrosion threshold usually occurs at 0.03% (300 ppm) chloride concentration by weight of concrete.

Extracted from Ref. 2

Fig. 11 - Total Chloride Content vs Depth from Vertical Beam Surface



An epoxy mortar strip bonded to a concrete slab, as constructed.

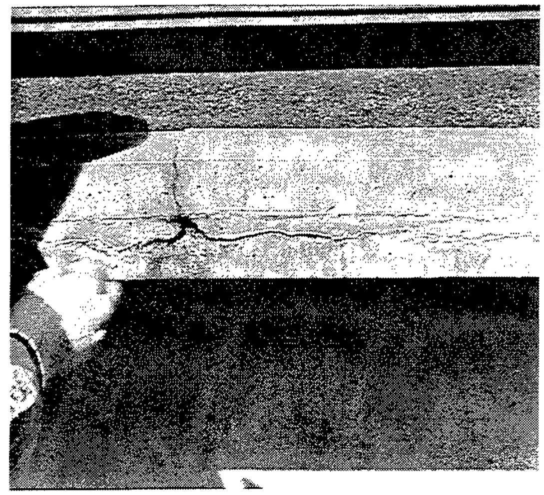
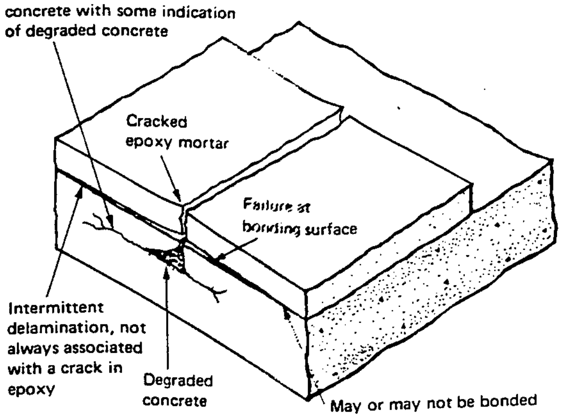


Fig. 12-B Concrete railroad platform with epoxy mortar curb showing concrete deterioration.



Condition of the same concrete and epoxy mortar after several years **(Ref. 10)**

Fig. 12-A

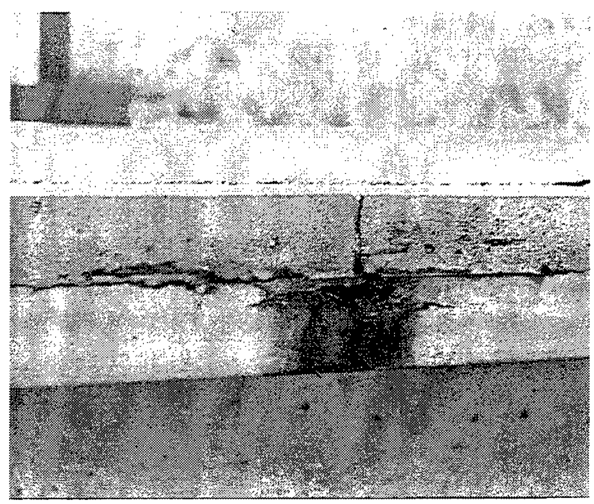


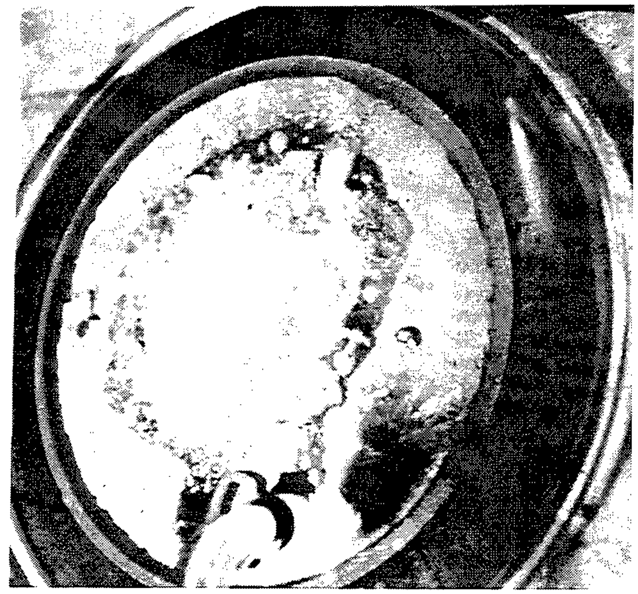
Fig. 12-C Concrete railroad platform with epoxy mortar curb showing concrete deterioration.

INTERACTION OF EPOXY MORTAR CURB ON CHLORIDE CONCRETE RAILROAD PLATFORM

FIG. 12

NONE BONDED - MORTAR PERMITTED WATER ACCESS TO TENDON ANCHOR WITH RESULTING CORROSION

FIG. 13-A



POROUS MORTAR USED IN POCKET DETECTED BY A BUBLLE VACUUM TEST

FIG. 13-B

SINGLE STRAND ANCHORAGE POCKETS

FIG. 13

COLD JOINT IN EXTERIOR-ANCHORAGE PROTECTION ON A BRIDGE IN NORTHEAST UNITED STATES

FIG. 14

APPENDIX A

Report 2

Durability and Behavior of Prestressed Concrete Beams - Posttensioned Concrete Investigation Progress to July 1966

by E. C. Roshore

March 1967

PART II: MATERIALS AND MIXTURES

Materials

Portland cement

5. The portland cement used was a type III, high-early-strength cement (RC-480). The cement met all of the chemical and physical requirements[5] of a low-alkali, type III cement and is classified as having moderate sulfate resistance (tricalcium aluminate content of 8 percent).

Air-entraining admixture

6. A commercially prepared resin solution was used as the air-entraining admixture.

Aggregates

7. Manufactured limestone sand and crushed limestone coarse aggregate from a quarry near Nashville, Tenn., were used in the concrete mixtures; the manufactured limestone sand was also used in the mortar mixture. A natural sand from Georgetown, Miss., was used in one grout mixture (beam 14).

Membrane-forming curing compound

8. A commercially available, white, membrane-forming curing compound was used to prevent excessive loss of moisture from the test beams during their curing in wooden forms.

Epoxy-resin compounds

9. Two commercially available epoxy-resin compounds (grout and binder) were used to protect the ends of the posttensioning tendons from corrosion.

Posttensioning tendons and accessories

10. Commercially available posttensioning tendons and accessories, representing four well-known posttensioning systems, systems A, B, C, and D, were used.

Mixtures

Portland-cement concrete mixtures

11. Beams proper. The concrete mixtures used for the test beams proper (i.e. excluding grout and anchorage protection) had the following characteristics:

Cement	Max Size Aggregate in.	Air Content %	Water-Cement Ratio (by Wt)	Slump in.	Cement Factor bags/ cu yd	Nominal Compressive Strength psi (28 Days Age)
Type III (high-early-strength)	3/4	4.0-5.0	0.52 (5.85 gal/bag)	1-1/2-2	5.98-6.05	6000

Mixture and test data for each batch of this concrete are given in table 1.

12. End protection. The concrete mixtures used for end-anchorage protection (excluding epoxy concrete) had the following characteristics:

Cement	Max Size Aggregate in.	Air Content %	Water-Cement Ratio (by Wt)	Slump in.	Cement Factor bags/ cu yd	Nominal Compressive Strength psi (28 Days Age)
Type III (high-early-strength)	3/4	3.5-5.0	0.80 (9.03 gal/bag)	1-1/4-2	3.90-3.96	3000

Mixture and test data for each batch of this concrete are given in table 2.

Epoxy-concrete mixtures

13. The epoxy concrete used for end-anchorage protection had the following characteristics:

Cement	Max Size Aggregate, in.	Mixture Proportions (by Wt), Epoxy Binder:Sand:Coarse Aggregate	Compressive Strength, psi (28 Days Age)
None	3/4	2.83:7.00:10.00	9,320 - 11,320

Test data for each batch of this concrete are given in table 2.

Mortar mixtures

14. The mortar mixtures used for end-anchorage protection had the following characteristics:

Cement	Max Size Aggregate	Water-Cement Ratio (by Wt)	Cement Factor bags/cu yd	Compressive Strength, psi (28 Days Age)
Type III (high-early-strength)	100% passing No. 4 sieve	0.44 (4.97 gal/bag)	10.90	7710-7800

Each mortar mixture also contained a small amount of aluminum powder. Test data for each mortar batch are given in table 2.

Grout mixtures

15. The steel tendon in 19 of the 20 test beams was pressure-grouted after posttensioning, using a grout of the following characteristics:

Cement	Water-Cement Ratio (by Wt)	Compressive Strength, psi (7 Days Age)	Linear Expansion, % (3 Days Age)
Type III (high-early-strength)	0.40-0.49 (4.51-5.53 gal/bag)	3740-6430	0-7

All grouts were neat cement grouts except that used for beam 14, which was a natural sand grout (100 percent passing No. 30 sieve). All of the grouts contained a small amount of aluminum powder (1 to 3 grams per bag of cement). Additional information on grout mixtures is given in table 3.

PART III: TEST SPECIMENS

Types of Test Specimens

16. Three hundred test specimens were made from 68 batches of either concrete, mortar, or grout mixtures, as shown in the following tabulation:

Type Mixture	Total No. of Batches Made	Specimen Size, in.	Type Specimen	No. of Specimens	No. of Batches Represented by Specimens
Air-entrained concrete, 6000 psi	40	10 by 16 by 96	Beam	20	40
		6 by 12	Cylinder	240	
Air-entrained concrete, 3000 psi	15	6 by 12	Cylinder	15	15
Epoxy concrete	6	3 by 6	Cylinder	6	6
Mortar	2	3 by 6	Cylinder	4	2
Grout	19	2	Cube	15	5
Total	82			300	68

Note: These specimens were fabricated in accordance with applicable portions of test methods in the Handbook for Concrete and Cement.[5]

Test Beams

Description

17. The 20 air-entrained concrete beams were rectangular at the ends (10- by 16-in. cross section) and were 96 in. long with a 68-in.-long thin web section (5- by 6-in. cross section). Plate 1 gives the dimensional details. Nineteen of the beams contained one flexible metal tube (1-1/4- to 1-5/8-in. OD); the other beam contained one unbonded, coated post-tensioning tendon (system C), which was spiral wrapped with paper. The coating material consisted of a nonvolatile calcium soap and mineral oil grease with a rust preventing additive. All beams contained other steel reinforcing which was provided with a nominal 3/4 in. of concrete cover. The eccentricity of the unbonded tendon or the metal tubing which was to enclose the posttensioning tendon was either 0, 1, 2, or 3 in. The other reinforcing was longitudinal steel, stirrups, and bearing grillages for

the anchorages. Table 4 gives details concerning the reinforcement used.

Fabrication of test beams

18. The test beams were molded in wooden forms in the laboratory; each beam was made from two 7-cu-ft batches of concrete. Electric vibrators were used to consolidate the concrete; the dates on which the beams were cast are given in table 5. The top surface of each beam was coated with a white membrane-forming curing compound; the other surfaces of the beams were protected during the curing period by the wooden forms. After 3 days, the forms were removed and the beams were subjected to 10 to 11 days of water curing.

Posttensioning of beams

19. When the concrete was 13 to 21 days old (table 5), all beams were posttensioned. In 19 of the 20 beams the posttensioning was accomplished by inserting a steel tendon through the metal tube, anchoring the tendon at the ends, and jacking against the concrete. In the other beam the steel tendon which had been cast into the beam was stressed against the concrete. The posttensioning force used was in accordance with the recommendations of the supplier of the particular posttensioning system used; details of the four systems are given in the following tabulation.

System	No. of Beams Made	Type of Tendon	Method of Anchoring	Initial Posttensioning Force, tons	Estimated Final Posttensioning Force, tons
A	6	12 steel wires (each 0.196-in. diam)	Wedge action	42	23
B	6	1 steel bar (7/8-in. diam)	Direct bearing	35	26
C	4*	8 steel wires (each 1/4-in. diam)	Direct bearing	35	30
D	4	12 steel wires (each 1/4-in. diam)	Direct bearing	50	42

* One of these tendons was an unbonded, coated tendon (beam 13).

End anchorages

20. The end anchorages of the tendons were of either the external or flush type. The external anchorage components were placed outside of the beam on the ends (photograph 2), while the flush anchorage assembly was

recessed in the ends of the beams (photograph 3). Posttensioning systems C and D had external anchorages only, while systems A and B had both external and flush anchorages (table 6).

Grouting of tendons

21. The steel tendon in 19 of the 20 beams was pressure-grouted (100-psi pressure*) when the concrete was 17 to 40 days old (table 5). Eighteen of the grouts were neat cement grouts; the other was a natural sand grout (beam 14). All grouts contained a small amount of aluminum powder to prevent shrinkage.

End-anchorage protection

22. After the tendons had been grouted, end-anchorage protection was placed on the ends of the beams when the concrete was 32 to 48 days old (table 5). The end protection for the external anchorages consisted of cover which formed an extension of the rectangular end section of the beam. The flush anchorage protection was placed by using a wooden form against the end of the beam and merely filled the recess in the end of the beam. Twelve types of end-anchorage protection were used to provide at least 1-1/2 in. of cover for the end-anchorage components of all tendons. The cover consisted of either air-entrained concrete, sand-cement mortar, or epoxy concrete. Table 7 lists details of the various types of end protection used. It should be noted that some of the end protections contained reinforcing steel which was provided with a nominal 3/4 in. of cover.

Auxiliary Specimens

23. The 280 auxiliary specimens were made indoors using metal molds. Two hundred and forty 6-in.-diam by 12-in.-high cylinders represented the 40 air-entrained concrete batches (6 per batch) used for the test beams. Fifteen 6- by 12-in. cylinders represented the 15 air-entrained concrete batches used for end protection. Six 3- by 6-in. epoxy-concrete cylinders represented 6 batches of epoxy concrete used for end protection. Four 3- by 6-in. mortar cylinders represented 2 mortar batches used for end protection. Fifteen 2-in. cubes represented 5 of the 19 grout mixtures.

---

* Only 50-psi pressure could be obtained on beam 1.

PART IV: TESTS AND RESULTS

Tests of Freshly Mixed Concrete

Test beams proper

24. The slump, air content, and bleeding of each batch of concrete used in the test beams proper were determined using appropriate test methods.[5] Test results are given in table 1. Concrete slumps ranged from 1-1/2 to 2 in.; air contents ranged from 4.0 to 5.0 percent. Bleeding varied from 0.0 to 1.2 percent.

End protection

25. The slump and air content of each batch of portland-cement concrete used for end protection were determined using appropriate test methods.[5] Test results are given in table 2. The slump of these mixtures ranged from 1-1/4 to 2 in. Air contents varied from 3.5 to 5.0 percent. No tests were made on the freshly mixed batches of epoxy concrete used for end protection.

Compressive Strength Tests of Concrete Mixtures

26. Compressive strength tests were conducted using appropriate test methods[5] on test cylinders representing each of the concrete mixtures used.

27. The compressive strength of the portland-cement concrete mixtures used in the test beams proper was determined at 7, 14, and 28 days age; the test results ranged from 4320 to 7010 psi and are given in table 1. The average compressive strength obtained at each age is shown below.

	Average Compressive Strength, psi, at Age Shown		
	7 days	14 days	28 days
Portland-cement concrete mixtures (beam proper)	4935	5450	5935

28. The compressive strength of the portland-cement-concrete mixtures used for end protection was determined at 28 days and at 1 year age; these results are given in table 2. The average 28-day compressive strength was 3360 psi; the values ranged from 2510 to 3680 psi. The average 1-year strength was 4140 psi; the values ranged from 3880 to 4460 psi.

29. The compressive strength of the epoxy-concrete mixtures used for end protection was determined at 28 days age; these test results are given in table 2. The average 28-day strength was 10,430 psi; the values ranged from 9320 to 11,320 psi.

Tests of Mortar Mixtures

30. Compressive strength tests were conducted at 28 days age using appropriate test methods[5] on test cylinders representing each of the mortar mixtures used for end protection. The average 28-day strength was 7755 psi; the values ranged from 7710 to 7800 psi; these test results are given in table 2.

Tests of Grout Mixtures

31. The 7-day compressive strength and the 3-day expansion (using micrometer bridge) of the grout mixtures used were determined in accordance with appropriate test methods.[5] The average 7-day compressive strength of the neat cement grouts was 5080 psi; the values ranged from 4480 to 6430 psi. The 7-day compressive strength of the one natural sand grout was determined to be 3740 psi. The 3-day linear expansion values obtained on the neat cement grout mixtures ranged from 0 to 7 percent. The 3-day expansion value obtained on the natural sand grout was 2 $\pm$ 1 percent. All of these data are given in table 3.

Camber Measurements

32. The midpoint deflection (camber) of each of the test beams resulting from the posttensioning operation was measured using dial gages. Two dial gages were used for each camber measurement; one gage (least reading 1/10,000 in.) was mounted on either side of the concrete beam at the center. The results of the camber measurements, rounded to the nearest 1/1000 in., are given in table 8.

33. The deflections obtained ranged from 0.000 to 0.008 in. The test results indicate, as should be the case, that camber increases with

increased eccentricity of the posttensioning tendon (see tabulation below).

Eccentricity, in.	Average Camber, in. (Average of 5 Beams)
0	0.001
1	0.003
2	0.006
3	0.007

Field Exposure Tests

34. The 20 posttensioned beams were installed at half-tide elevation on the beach at Treat Island in June 1961. Since 1961, the beams have been subjected to 623 cycles of tidal freezing and thawing (see paragraph 4).

Visual inspection

35. The posttensioned beams were visually inspected in the summers of 1962, 1963, 1964, and 1966 by a panel of qualified observers. The beams were rated by the observers in accordance with a set of instructions; a sample inspection sheet with instructions is given in Appendix A. The condition of each test beam, based on the inspection notations made by the observers, was expressed numerically, using the scoring system outlined in Appendix B. These numerical ratings are given in table 9. The numerical ratings for the types of end protection are also given in table 9. Numerical ratings for end protection are generally lower than the ratings for the whole beam because only those elements pertaining to ends of beams are summed to obtain the ratings for end protection (Appendixes A and B).

Pulse velocity tests

36. Pulse velocity measurements were made on all beams prior to their installation at the Treat Island Exposure Station. These measurements were made in accordance with applicable provisions of test method CRD-C 51-57.[5] Additional pulse velocity measurements are being taken annually while the test beams are being subjected to severe weathering. The readings are being taken by transmitting a sound pulse through the test beam along its long axis (longitudinally) and also through the thin web section (transversely). The test instrument[5] measures the time of travel of the sound pulse, and from the time of travel and the path length,

values for the velocity of sound in the concrete (V) are obtained. The square of the velocity thus determined is expressed as a percentage of the square of the initial velocity obtained at installation ($\%V^2$).[4]

37. The transverse pulse velocity values ($\%V^2$) obtained to date (table 8) are erratic and do not provide a satisfactory indication as to the condition of the test beams. This is possibly due to the short path length (5 in.) used in making these readings. The longitudinal pulse velocity values ($\%V^2$) appear to be reasonably consistent and do indicate some deterioration in the beams since eight of the 1965 longitudinal ($\%V^2$) readings are below 100%. The results of pulse velocity measurements made in 1966 are not given in table 8 since the readings obtained were unsatisfactory because of an equipment malfunction.

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2. __________, Durability and Behavior of Pretensioned-Prestressed Concrete Beams, by E. C. Roshore. Miscellaneous Paper No. 6-611, Vicksburg, Miss., December 1963.

3. __________, Durability of Prestressed Concrete Beams, by E. C. Roshore. Miscellaneous Paper No. 6-665, Vicksburg, Miss., July 1964.

4. __________, Investigation of Performance of Concrete and Concreting Materials Exposed to Natural Weathering, by E. C. Roshore. Technical Report No. 6-553, vol 1, with annual supplements, Vicksburg, Miss., June 1960.

5. __________, Handbook for Concrete and Cement, with quarterly supplements. Vicksburg, Miss., August 1949.

Table 2

Characteristics* of Mixtures Used for End Protection

Beam No.	Type Protection	Cement Factor bags/ cu yd	Slump in.	Air Content %	Compressive Strength, psi, at Ages Shown 28 days	1 year
1	Concrete (both ends)	3.94	1-1/2	4.0	3,680**	--
2	Concrete (both ends)	3.94	1-1/2	4.0	3,410**	--
3	Concrete (both ends)	3.94	1-1/2	4.0	3,500**	--
4	Epoxy concrete (both ends)	--	--	--	10,750†	--
5	Concrete (both ends)	3.96	1-1/2	3.5		3,880**
6	Mortar (one end)	10.90	--	--	7,710†	--
	Epoxy concrete (other end)	--	--	--	10,400†	--
7	Concrete (both ends)	3.90	1-3/4	5.0	3,450**	--
8	Concrete (both ends)	3.94	1-1/2	4.0	3,500**	--
9	Concrete (both ends)	3.90	2	5.0	2,510**	--
10	Concrete (both ends)	3.92	1-1/2	4.3	--	4,120**
11	Epoxy concrete (both ends)	--	--	--	10,930†	--
12	Mortar (one end)	10.90	--	--	7,800††	--
	Epoxy concrete (other end)	--	--	--	9,320†	--
13	Concrete (one end)	3.90	1-3/4	5.0	3,090**	--
	Concrete (other end)	3.91	1-3/4	4.6	3,570**	--
14	Concrete (both ends)	3.93	1-1/2	4.2	3,430**	--
15	Concrete (both ends)	3.94	2	4.0	--	4,110**
16	Epoxy concrete (both ends)	--	--	--	11,320†	--
17	Concrete (both ends)	3.90	1-1/2	5.0	3,480**	--
18	Concrete (both ends)	3.94	2	4.0	--	4,460**
19	Concrete (both ends)	3.90	1-1/4	5.0	3,340**	--
20	Epoxy concrete (both ends)	--	--	--	9,870†	--

Note: Water-cement ratio for each portland-cement concrete mixture was 0.80 by weight. Water-cement ratio for each mortar mixture was 0.44 by weight. The epoxy-concrete mixtures contained no cement.

* See reference 5 in Literature Cited for test methods.

** Compressive strength of one 6- by 12-in. cylinder at age shown.

† Compressive strength of one 3- by 6-in. cylinder at age shown.

†† Average compressive strength of three 3- by 6-in. cylinders at age shown.

Table 3

Grout Mixtures

Beam No.	Type Grout*	Water-Cement Ratio (by Wt)	Linear Expansion, %**	Compressive Strength psi (7 days Age)†
1	Neat cement	0.40	2 ± 1	--
2	Neat cement	0.40	--	--
3	Neat cement	0.40	2 ± 1	4480
4	Neat cement	0.40	6 ± 1	4700
5	Neat cement	0.40	6 ± 1	4700
6	Neat cement	0.40	--	--
7	Neat cement	0.40	--	--
8	Neat cement	0.40	1 ± 1	--
9	Neat cement	0.40	--	--
10	Neat cement	0.40	--	--
11	Neat cement	0.40	--	--
12	Neat cement	0.40	1 ± 1	--
13	None; unbonded, coated tendon; not grouted	--	--	--
14	Natural sand††	0.49	2 ± 1	3740
15	Neat cement	0.40	--	--
16	Neat cement	0.40	--	--
17	Neat cement	0.40	--	--
18	Neat cement	0.40	2 ± 1	6430
19	Neat cement	0.40	--	—
20	Neat cement	0.40	--	—

* All grouts contained a small amount of aluminum powder.

** Percent after final set, 3 days age; measured with micrometer bridge; test specimen was a 6-in.-long cylinder, 3 in. in diameter.

† Each strength given is average of strengths of three 2- by 2-in. cubes; see reference 5 in Literature Cited for test method.

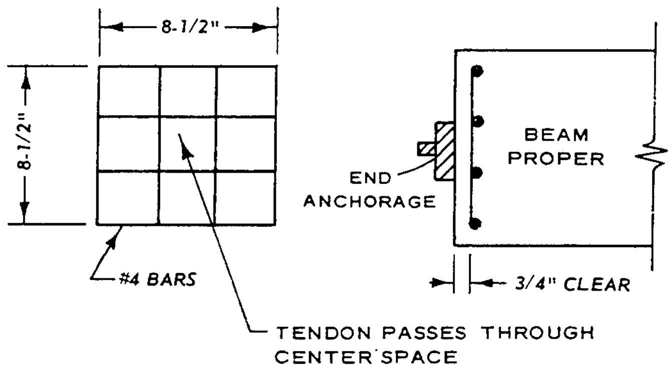
†† This grout contained natural sand, 100% passing the No. 30 sieve.

Table 4

Reinforcing Steel Used in Test Beams

Description	No. Used per Beam	Consisting of:
Bearing grillage	2	Eight 1/2-in.-diameter deformed steel bars (see note 1 below)
Stirrups	8	Two 3/8-in.-diameter deformed steel bars
Longitudinal bars	4	1/2-in.-diameter deformed steel bars
Longitudinal tie bars	2	1/4-in.-diameter deformed steel bars
End-protection reinforcing (see note 2 below)	2	3/8-in.-diameter deformed steel bars

Note 1: The bearing grillages were located 3/4 in. from the end anchorage; each grillage consisted of four bars each way as shown.



Note 2: End-protection reinforcing was used on only 12 beam ends. This 3/8-in.-diameter reinforcing lapped with the 1/2-in.-diameter longitudinal bars in the beam proper for a length of 12 in. as shown below.

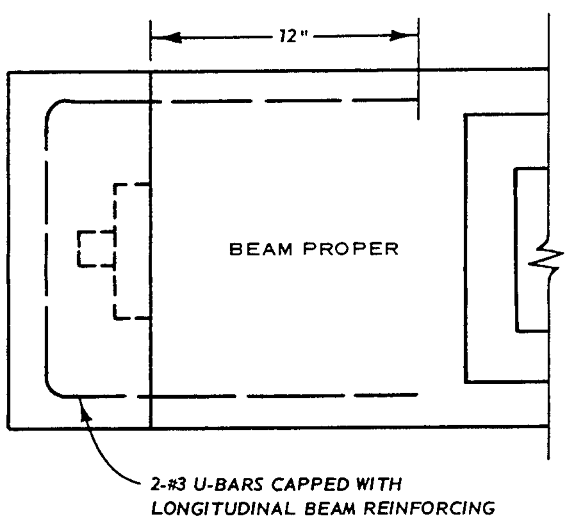


Table 5

Schedule of Operations for Posttensioned Beams

Beam No.	Casting Date	Age of Concrete (days) in Test Beams Proper at Time of: Stressing of Tendon	Grouting of Tendon	Placement of End Protections
1	9 Dec 1960	14	19	33
2	27 Jan 1961	14	19	34
3	21 Oct 1960	14	17	33
4	2 Dec 1960	14	18	33
5	25 Nov 1960	14	25	33
6	28 Oct 1960	13	18	32 and 33
7	6 Jan 1961	14	18	33
8	7 Oct 1960	19	24	32
9	13 Jan 1961	17	18	33
10	20 Jan 1961	14	26	35
11	16 Dec 1960	21	25	33
12	14 Oct 1960	14	17	32 and 33
13	23 Sept 1960	14	Not grouted	32 and 33
14	30 Sept 1960	14	18	32
15	3 Mar 1961	14	20	41
16	4 Nov 1960	14	17	33
17	17 Feb 1961	14	34	42
18	24 Feb 1961	14	40	48
19	10 Feb 1961	14	20	36
20	3 Feb 1961	20	27	35

Table 6

General Information, Posttensioned Beams at Treat Island

(Installed June 1961)

Beam No.	Post-tensioning System	Eccentricity of Tendon in.	Estimated Final Posttensioning Force, tons	Type of End Protection (See Note) Landward End	Seaward End
1	A	0	23	Flush (1)	Ext (5)
2	A	0	23	Ext (4)	Ext (2)*
3	A	3	23	Ext (3)	Ext (1)
4	A	2	23	Ext (7)	Flush (7)*
5	A	2	23	Ext (6)	Flush (6)
6	A	1	23	Flush (9)	Ext (8)
7	B	0	26	Ext (1)*	Flush (1)
8	B	2	26	Ext (2)*	Ext (4)*
9	B	3	26	Ext (3)*	Ext (5)*
10	B	3	26	Flush (6)	Ext (6)
11	B	1	26	Flush (7)	Ext (7)
12	B	1	26	Ext (8)	Flush (9)
13**	C	0	30	Ext (1)*	Ext (3)*
14	C	1	30	Ext (2)*	Ext (4)*
15	C	3	30	Ext (5)	Ext (6)*
16	C	2	30	Ext (7)	Ext (8)
17	D	3	42	Ext (1)†	Ext (3)†
18	D	0	42	Ext (4)*	Ext (2)*
19	D	2	42	Ext (5)††	Ext (6)†
20	D	1	42	Ext (8)	Ext (7)*

Note: Concrete placed against a <u>cold joint</u> with <u>no</u> surface treatment and <u>no</u> reinforcement. [Ext (1) and Flush (1)]

Concrete placed against a <u>cold joint</u> with <u>no</u> surface treatment but <u>with</u> reinforcement. [Ext (2)]

Concrete placed against a <u>bush-hammered</u> surface and with <u>no</u> reinforcement. [Ext (3)]

Concrete placed against a <u>bush-hammered</u> surface but <u>with</u> reinforcement. [Ext (4)]

Concrete placed against a surface which had been treated with a <u>retarding agent</u> and <u>no</u> reinforcement. [Ext (5)]

Concrete bonded to the ends of the beam with an <u>epoxy adhesive</u> and <u>no</u> reinforcement. [Ext (6) and Flush (6)]

<u>Epoxy concrete without</u> reinforcement. [Ext (7) and Flush (7)]

<u>Epoxy concrete with</u> reinforcement. [Ext (8)]

<u>Sand-cement mortar</u> with <u>aluminum powder</u> additive, comparatively dry and well tamped. [Flush (9)]

* End protection has developed a crack at bond line.

** Tendon in this beam was an unbonded, coated tendon (<u>not</u> grouted).

† End protection has become detached.

†† End protection has become loose.

Table 7

Twelve Types of End-Anchorage Protection Used for Posttensioned Beams

Type No.	Designation*	End-Anchorage Protection: Protective Material Used	End-Anchorage Protection: Beam-End Surface Treatment	End-Anchorage Protection: Steel Reinforcement	No. of Beam Ends
1	Ext (1)	Air-entrained concrete	None, cold joint	No	4
2	Flush (1)	Air-entrained concrete	None, cold joint	No	2
3	Ext (2)	Air-entrained concrete	None, cold joint	Yes	4
4	Ext (3)	Air-entrained concrete	Bush-hammered	No	4
5	Ext (4)	Air-entrained concrete	Bush-hammered	Yes	4
6	Ext (5)	Air-entrained concrete	Retarded	No	4
7	Ext (6)	Air-entrained concrete	Epoxy-coated (sandblasted)	No	4
8	Flush (6)	Air-entrained concrete	Epoxy-coated (sandblasted)	No	2
9	Ext (7)	Epoxy concrete	Sandblast and primer	No	4
10	Flush (7)	Epoxy concrete	Sandblast and primer	No	2
11	Ext (8)	Epoxy concrete	Sandblast and primer	Yes	4
12	Flush (9)	Sand-cement mortar (with aluminum powder)	Sandblasted	No	2
				Total beam ends	40

* See table 6.

Table 8

Camber and Pulse Velocity Test Results
(Beams Installed at Treat Island in June 1961)

				1961 (Initial)				Treat Island Exposure Station							
								1962 89 Cycles††		1963 195 Cycles††		1964 330 Cycles††		1965 493 Cycles††	
Beam No.	Post-tension-ing System	Eccen-tricity of Tendon in.	Midpoint Deflection of Beam (Camber) During Post-tension-ing, in.*	Trans-verse Pulse Veloc-ity (V) fps**	% V^2	Longi-tudinal Pulse Veloc-ity (V) fps†	% V^2	Trans-verse % V^2	Longi-tudi-nal % V^2	Trans-verse % V^2	Longi-tudi-nal % V^2	Trans-verse % V^2	Longi-tudi-nal % V^2	Trans-verse % V^2	Longi-tudi-nal % V^2
1	A	0	0.002	15,000	100	14,295	100	116	116	‡	118	134	111	134	121
2	A	0	0.000	17,375	100	15,020	100	84	104		106	213	103	134	101
3	A	3	0.007	16,040	100	14,435	100	117	108		109	122	106	145	96
4	A	2	0.004	17,670	100	14,435	100	113	112		115	231	111	126	112
5	A	2	0.006	15,795	100	14,735	100	117	105		110	122	104	158	72
6	A	1	0.003	17,090	100	14,610	100	100	107		109	146	106	149	115
7	B	0	0.001	17,375	100	14,760	100	83	104		109	196	104	131	80
8	B	2	0.004	16,290	100	14,575	100	98	104		102	140	93	149	95
9	B	3	0.007	17,230	100	14,825	100	102	109		108	176	102	111	73
10	B	3	0.006	17,670	100	15,105	100	95	105		106	173	105	129	103
11	B	1	0.001	18,450	100	15,160	100	100	101		105	182	99	128	102
12	B	1	0.002	17,820	100	14,840	100	88	100		105	152	102	104	83
13	C	0	0.003	16,680	100	16,120	100	103	85		103	152	99	118	98
14	C	1	0.003	17,230	100	14,720	100	83	94		103	157	98	111	146
15	C	3	0.006	17,975	100	14,625	100	111	95		107	178	103	122	107
16	C	2	0.007	17,670	100	14,770	100	119	103		108	156	103	154	113
17	D	3	0.008	17,670	100	14,790	100	100	78		99	153	91	139	130
18	D	0	0.000	17,820	100	14,020	100	70	81		113	123	87	124	83
19	D	2	0.008	18,785	100	14,950	100	88	107		92	176	104	112	107
20	D	1	0.005	18,615	100	14,765	100	98	105		107	192	103	114	109

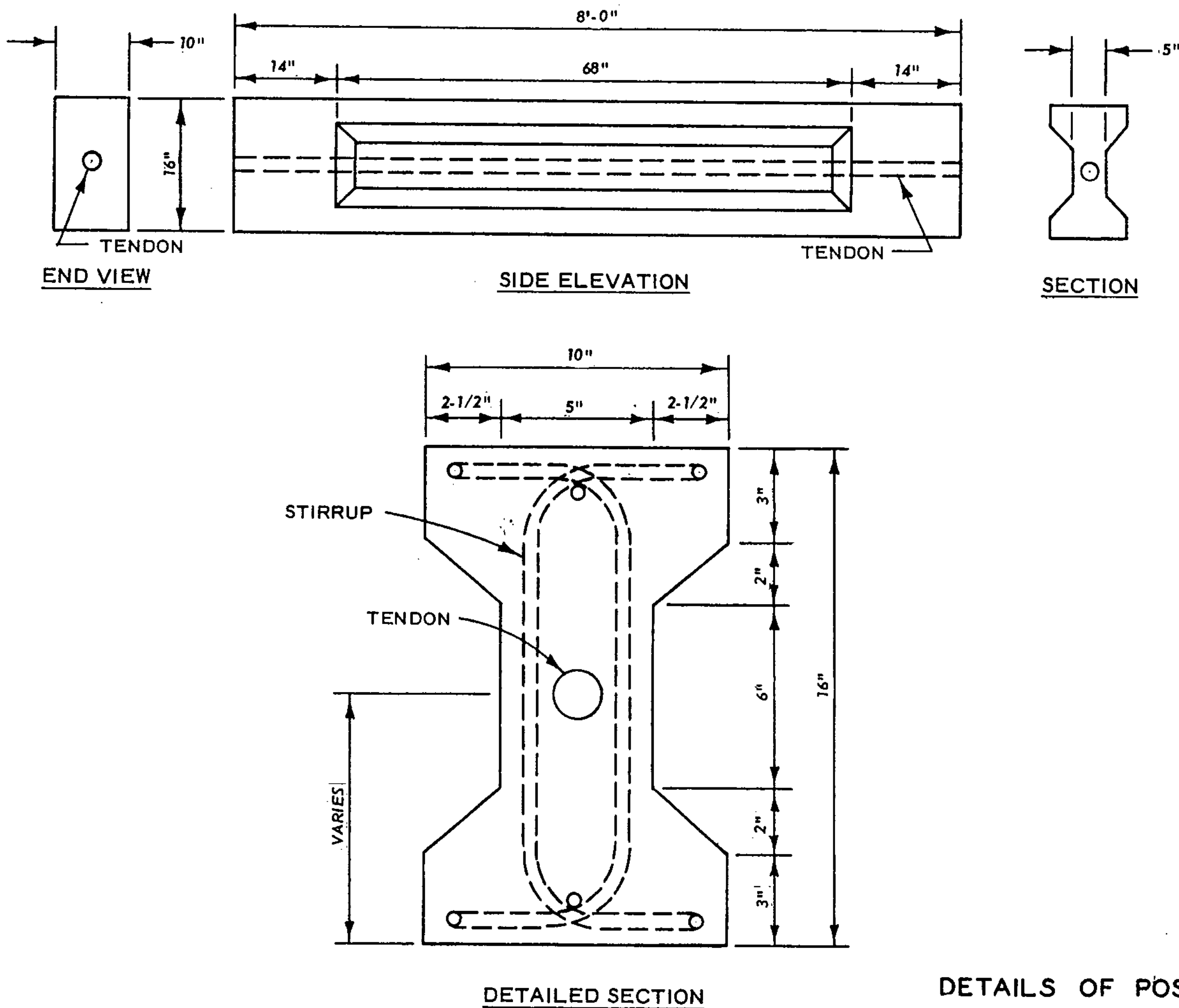
Note: V is the initial velocity of sound in the concrete, measured at time of installation at exposure station.
* Deflection expressed to nearest 0.001 in.
** Pulse velocity readings taken through thin web section of beam.
† Pulse velocity readings taken longitudinally through entire length of beam.
†† Cycles of tidal freezing and thawing.
‡ 1963 transverse readings were not satisfactory because of equipment malfunction.

Table 9

Numerical Rating* of Posttensioned Beams Exposed at Treat Island, Maine (1961-1966)

Beam No.	Numerical Rating of Beam** 0 Cycles 1961	89 Cycles 1962	195 Cycles 1963	330 Cycles 1964	623 Cycles 1966	End Protection Designation†	No. of Beam Ends	Avg Numerical Rating of End Protection** 0 Cycles 1961	89 Cycles 1962	195 Cycles 1963	330 Cycles 1964	623 Cycles 1966
1	0	17	18	25	35	Ext (1)	4	0	3	6	7	11
2	0	11	18	24	22	Flush (1)	2	0	0	0	1	2
3	0	18	23	24	38	Ext (2)	4	0	4	6	10	11
4	0††	19	20	24	29	Ext (3)	4	0	1	4	10	10
5	0	17	28	25	28	Ext (4)	4	0	5	6	11	13
6	0	12	22	17	31	Ext (5)	4	0	1	1	3	2
7	0††	7	19	20	29	Ext (6)	4	0	2	4	9	12
8	0‡	20	31	45	68	Flush (6)	2	0	0	0	0	0
9	0††	18	23	24	41	Ext (7)	4	0	0	0	1	1
10	0††	14	21	19	25	Flush (7)	2	0	1	1	2	2
11	0††	8	13	12	22	Ext (8)	4	0	0	0	1	4
12	0††	16	25	24	41	Flush (9)	2	0	0	0	0	2
13	0††	11	12	16	30							
14	0††	25	38	40	48							
15	0††	15	18	17	25							
16	0††	10	15	13	19							
17	0	16	46	50	74							
18	0††	8	16	22	39							
19	0††	12	15	41	48							
20	0††	10	15	16	25	Total	40					

* See Appendix B for scoring; 0 = perfect condition.
** Beams not rated in 1965.
† See table 6.
†† These beams were chipped in several places during shipment and placement.
‡ This beam was chipped in several places during shipment and placement, which resulted in exposure of 3 in. of reinforcing.



DETAILS OF POSTTENSIONED CONCRETE BEAM

APPENDIX B

Report 4

Durability and Behavior of Prestressed Concrete Beams - Posttensioned Concrete Beam Investigation with Laboratory Tests from June 1961 to September 1975

by Edward F. O'Neil

February 1977

PART IV: SUMMARY OF OBSERVATIONS AND TESTS

Ultimate Tensile Strength and Elastic Properties of the Strands

139. Each strand of prestressing steel was analyzed by tension testing methods to determine its ultimate tensile strength, total elongation, and stress-strain properties. The tests were conducted in general accordance with the applicable portions of ASTM Designation A 370-68[5] for testing, and the results were compared with ASTM Designation A 421-59T, which was the specification that was current at the time the steel was manufactured. Structural testing results are presented in Table 5.

140. As preparation for testing, each strand was scraped and sanded to remove all the products of corrosion and then measured to determine the diameter. Measurements were taken at 2-in. intervals for approximately 12 in. on each end of the strand, and then measurements were taken at 6-in. intervals on the remainder of the strand. For structural testing, three 12-in. segments of each strand were used, two of which were the 12-in. sections at each end of the strand. The third segment was cut from the remaining part of the strand where the minimum diameter was found. The data on the strand diameters are presented in Table 7.

141. Each strand was tested in tension to ultimate load. A load-deflection curve for each strand was also made. Representative stress-strain curves for the steel strands are found in Appendix A, Plates A5-A7.

Properties of the Grout

Grout density test

142. Nine samples of grout from each metal conduit, three from each end of the beam and three from the middle, were analyzed to determine the density of the grout. The sample densities were measured,

and results are presented in Table 8.

143. The densities of the grouts ranged from 118.20 lb/ft^3 for beam 11 to 125.82 lb/ft^3 for beam 6. This is a normal range for pressure-pumped grout. With the exception of the few areas where the grout contained bleed water and air voids, this grout was essentially sound and of moderate density.

pH of the grout

144. Samples of grout were taken from the landward end, midsection, and seaward end of the conduits of seven of the eight beams tested (beams 1, 3, 6, 9, 11, 15, and 19). These samples were pulverized and placed in distilled water for pH tests. The results of these tests are presented in Table 9. All the samples gave pH readings between 12.13 and 12.90, the lowest being from beam 19 and the highest from beam 3. These readings are all in the normal range of grout, which is in the neighborhood of 12.50.[6] Since sodium chloride would tend to lower the pH of portland cement and water, it was felt that any concentration of chlorides in the grout would be indicated by a lower than normal pH reading.

Summary of Strand Corrosion

145. Every strand that was examined during the testing period was found to be covered with different amounts and different kinds of corrosion. The intensity and quantity of these products have been cataloged in paragraphs 36-138. The degree and trend of rusting on the strands in this investigation indicated areas of heavy rusting and pitting and areas in which corrosion did not predominate. In the following paragraphs, the trends of each beam are summarized in accordance with data in Table 10.

146. The landward end of beam 3 showed that rust increased from light to heavy as it proceeded inward from the anchorage. The pitting was moderate to heavy and was heaviest on strands 1-6.

147. Unlike the landward end of beam 3, the seaward end did not exhibit any rusting pattern. The overall rusting of this end was

moderate; pitting was moderate to heavy, the heaviest concentrations being on strands 1-6, 10, and 12. The midsection of this beam was moderately rusted, with strands 9-12 heavily rusted at the landward end of the section. There was no orderly pattern of rusting. This beam was prepared for structural testing before tarnishing degrees could be obtained.

148. The single rod of beam 9 was lightly rusted. The heaviest rusting, which was moderate, occurred at the seaward end. Over the entire rod, the pitting and tarnishing were moderate.

149. Beam 13, the beam that was not grouted but covered with grease, showed the heaviest rusting at the landward end. Proceeding in from the landward end, the rust decreased from heavy to light. Most of the pitting and tarnishing was light, but strands 6-8 showed moderate amounts of both.

150. At the seaward end, the rusting showed a decreasing trend as it moved away from the end anchorage. The rusting was less severe than that at the landward end for all cases. The pitting was light on most of the strands as was the tarnishing, but no pattern was noted in its occurrence.

151. Most of the grease remained on the strands over the midsection of the beam. Here, the majority of rust was moderate, and the tarnishing and pitting were light.

152. At the landward end of beam 15, the rust started moderately. Further inward from the anchorage, it increased, was heaviest at about 14 in., and then decreased again, becoming light toward the center. The behavior of the pitting followed that of the rust, and the tarnish was essentially heavy at the ends and moderate in the middle.

153. At the seaward end, the rusting was light over all the strands. The pitting was light, and the tarnishing was moderate. Neither rusting, pitting, nor tarnishing showed any trend toward increasing or decreasing. The midportions of the strands were very similar to the seaward ends. The rusting and pitting were light, and the tarnishing was moderate over the entire surface.

154. For the landward end of beam 19, the rusting exhibited no

pattern. Most of the strands were lightly rusted, particularly strands 6-9, 11, and 12. Rusting of the lower numbered strands was light at the landward end and moderate farther away from the end. The pitting was moderate for the lower numbered strands and light for the higher numbered ones. The tarnishing was moderate throughout the entire end.

155. At the seaward end, the rusting was basically light over the 32 in. The pitting was moderate in the lower numbered bars and light in the higher numbered ones. The tarnishing on the strands at this end was moderate.

156. The analysis of corrosion to the strands in the second investigation produced results in agreement with those in the first investigation. The strands were all rusted, pitted, and tarnished to varying degrees, and the severity of strand corrosion exhibited no pattern. At the landward end of beam 1, the rusting was moderate over most of the end; and the pitting and tarnishing were heavy over the whole end. There seemed to be less pitting and tarnishing on strands 10-12 than on the rest of the strands.

157. At the seaward end of the beam, the rusting changed from light to moderate at approximately 12 in. from the end. The pitting and tarnishing were heavy over the whole end except for a few strands that had only moderate pitting and tarnishing. The rusting and pitting at the midsection of the beam were moderate, with the pitting and tarnishing on strands 1-3 heavy. The tarnishing on the rest of the strands varied from moderate to heavy.

158. The analysis of beam 6 was similar to that of beam 19. At the landward end, the rusting was moderate over the entire end. On this end of the strands, the pitting was basically moderate and the tarnishing basically heavy. For the seaward end, the summary of strand corrosion was more ordered. The rusting was light on the first 13 in. of the strands and then became moderate over the rest of the end. Similarly, the pitting progressed from light to moderate, and the tarnishing was basically heavy.

159. The midsection was moderately rusted over the whole section. The pitting ranged from light to heavy with no specific trends,

and the tarnishing was moderate on the higher numbered strands and heavy on the lower numbered ones.

160. The single rod of beam 11 was moderately rusted, pitted, and tarnished over the entire beam. The only exception to this was the heavy pitting and tarnishing at the landward end from 0 to 22 in.

161. It should be reemphasized here that the classifications light, moderate, and heavy describe percentages of surface area covered and not depth of corrosion. In this study, the strands for which the corrosion was labeled heavy did not have deep corrosion and deterioration of the metal, with the exception of one end of one beam.

162. The analysis of the strands revealed that water, oxygen, and a condition of the surface of the steel needed to destroy the passivating film all existed at the surface of the steel. These conditions are all necessary to cause rusting. The rust that was found on the strands was not concentrated in any one area of the conduit, nor was the severity of the corrosion greater in some areas than in others. This observation can be made regarding any of the seven beams that had grouted tendons. The only beam in which the severity of the corrosion was greater at the ends was the beam containing a paper conduit filled with grease.

163. During the analysis of the rust on the strands, it became apparent that wherever two or more strands of posttensioning wire were touching or where a strand touched the metal conduit, the area was heavily rusted. These heavily rusted areas were approximately as long as the two areas which were in contact, and they extended about 1/8 in. to both sides of the contact points. These areas were heavily rusted regardless of the amount of corrosion in the adjacent areas.

164. Based on the conditions present in this study, the most probable explanation for this contact rusting is the production of electrolytic currents between areas of high and low salt concentrations.[7] Within the conduit the concentration of salt in the grout was low; also, less water and air were present, and thus fewer hydrogen and oxygen ions. In the concrete surrounding the conduit, the salt concentration was high, and the hydrogen and oxygen ions were more numerous due to the availability of water and air. With the electrolytic current

flowing, the steel inside the conduit became anodic, releasing ferrous ions and depositing hydrogen ions at the cathode. The buildup of hydrogen ions at the cathode has a tendency to slow the reaction to a stop; however, if there is an abundance of oxygen ions at the cathode, they will demand the hydrogen and allow the deposition of further hydrogen ions to continue the galvanic reaction.

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13.ABSTRACT *(Maximum 200 words)*

To determine the most effective end anchorage protection for posttensioned beams, 20 air-entrained posttensioned beams were fabricated and placed at the Treat Island Severe Weather Exposure Station in 1961.

The beams were fabricated using 4 types of posttensioning systems with 12 different types of end-anchorage protection over external and recessed anchorages. End-anchorage protection was applied to the beams using six different types of joint preparation: bush-hammering, epoxy-resin adhesive on sandblasted surface, retarding agent, sandblasted, sandblasted with primer, and no preparation. The end protections were made from three different mixtures: portland-cement, epoxy-resin concrete, and sand-cement mortar.

The 20 posttensioned test beams have been inspected annually by the Corps and biennially by other interests. Eleven of the beams have been autopsied at the U.S. Army Engineers Waterways Experiment Station in 1973-74 and 1983. Now, with 33 years of exposure of nine beams and the autopsy data, this report has been prepared jointly by Mr. Morris Schupack, Schupack Suarez Engineers, Inc., and Mr. Edward F. O'Neil, U.S. Army Engineer Waterways Experiment Station. It assembles the data obtained, evaluates the data, summarizes the important findings related to durability, and makes recommendations for durable details.

(Continued)

14.SUBJECT TERMS

Corrosion	Long-term exposure	Prestressed concrete
Durability	Marine environment	Prestressed steel
End-anchor protection	Post-tensioning	Sheath

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The main findings are that the flush (recessed) anchorage protection using portland-cement concrete is the superior detail. The external portland-cement concrete anchorage protection, properly anchored with reinforcing steel with adequate concrete cover, is also an effective protection.