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TENSILE CRACK EXPOSURE TESTS

Report 3

LABORATORY EVALUATION OF SERIES "A" BEAMS WITH RESULTS FROM 1951 TO 1975

by

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January 1980

Report 3 of a Series

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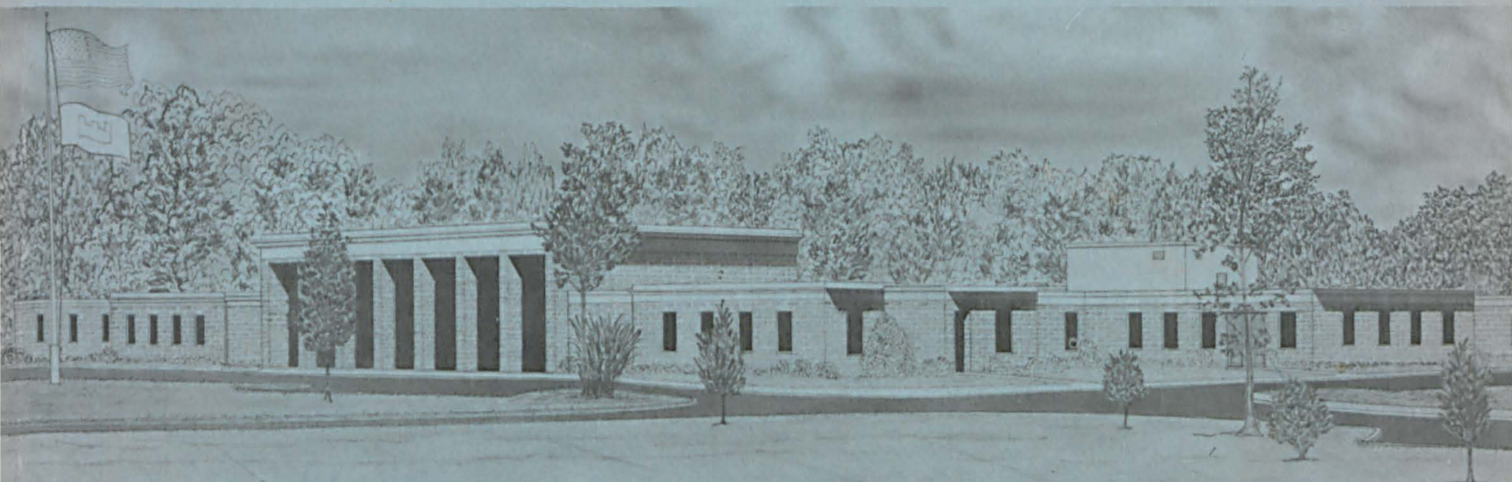

Prepared for Office, Chief of Engineers, U. S. Army
Washington, D. C. 20314

Under Civil Works Research Work Unit 010401/31276

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Concrete beams	Reinforced concrete
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Concrete exposure	Weathering (Concrete)

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A study was begun in 1950 to determine the effects of severe natural weathering to stressed, reinforced concrete beams of various compositions and degrees of stress. The objectives of the study were to obtain information on the long-term weathering of air-entrained and nonair-entrained concrete beams containing steels of different compositions, types of deformation, and having different levels of stress in the steel that caused varying degrees of cracking of the concrete.

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20. Abstract (Continued).

The beams were fabricated, cured, and loaded at the U. S. Army Engineer Waterways Experiment Station (WES) in 1951, then shipped to Eastport, Maine, and placed on the beach at the natural weathering exposure station on the south side of Treat Island, Cobscock Bay, Eastport, and Lubec. The beams were subjected to twice daily tidal cycles exposing them to wetting under considerable head, and drying to surface dry conditions. In addition, during the winter months, the beams were subjected to cycles of freezing and thawing with each tide when the air temperature was at or below 28°F (-2.2°C). The beams were inspected annually during the exposure period and evaluated by a team of inspectors rating the degree of deterioration. Each year, maximum crack widths were measured beginning in 1956 and continuing until 1975 when the exposure period was concluded after 25 years of weathering.

At the end of the exposure period, 13 of the 82 beams still remained at the testing site; 60 of the beams had been destroyed by freezing and thawing by 1956, since they were nonair-entrained concrete. Of the 13 beams remaining in 1975, 11 were returned to the laboratory for laboratory testing.

The results of the exposure study and the laboratory investigation indicated, among other findings, that stressing the steel to varying levels of stress over the exposure period did not reduce the ultimate moment carrying capacity of the beams; the maximum areas of corrosion occurred at spalled areas; corrosion to the steel could not be located at any flexural crack smaller than 0.015 in. (0.38 mm); and no area of maximum reduction of cross-sectional area due to corrosion occurred at the flexural cracks.

PREFACE

The investigation reported herein forms a part of Civil Works Research Work Unit 010401/31276 and was approved by Office, Chief of Engineers, in 2nd indorsement, dated 17 Jan 1951, to basic letter, dated 7 Dec 1950, subject: "Reinforced Concrete Beams for Tensile Crack Exposure Tests."

The test program was carried out by the U. S. Army Engineer Waterways Experiment Station (WES), under the direction of Messrs. Bryant Mather, Acting Chief, Structures Laboratory (SL), and J. M. Scanlon, Chief, Engineering Mechanics Division. This report was prepared by Mr. Edward F. O'Neil, Structures Branch, SL.

Directors and Commanders of the WES during the conduct of this investigation and the preparation and publication of this report were COL G. H. Hilt, CE, COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, CUSTOMARY INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Customary inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
bags* per cubic yard	55.76797972	kilograms per cubic metre
cal/g	4.184	kilojoules/kilogram
Fahrenheit degrees	5/9	Celsius degrees or Kelvins**
feet	0.3048	metres
inches	25.4	millimetres
pounds (mass)	0.4535924	kilograms
pounds (force)	4.448222	newtons
pound (force)-feet	1.355818	newton-metres
pounds (force) per square inch	0.006894757	megapascals
pounds (mass) per cubic yard	0.59327638	kilograms per cubic metre
square inches	645.16	square millimetres
tons (short)	907.1847	kilograms

* 94-lb bags of portland cement.

** To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9)(F - 32). To obtain Kelvin (K) readings, use: K = (5/9)(F - 32) + 273.15.

TENSILE CRACK EXPOSURE TESTS

LABORATORY EVALUATION OF SERIES "A" BEAMS WITH RESULTS FROM 1951 TO 1975

PART I: INTRODUCTION

Background

1. The information contained in this report concludes a portion of a study that was initiated in 1950 to determine the effects of severe natural weathering to stressed, reinforced concrete beams of various compositions and degrees of stress. The objectives of the study were to obtain information on the long-term weathering of air-entrained and nonair-entrained beams containing steels of different composition and deformations, having different stress levels that caused varying degrees of cracking of the concrete.

2. The series of beams described herein (Series A) was fabricated, cured, and loaded at the U. S. Army Engineer Waterways Experiment Station (WES) in 1951, then shipped to Maine and placed on the beach at the natural weathering exposure station on the south side of Treat Island, which is located in Cobscock Bay between Eastport and Lubec. The beams were subjected to twice daily tidal cycles (the average tidal range is 18 ft,* occasionally reaching maximums of 30 ft) exposing them to wetting under considerable head, and drying to surface dry conditions (this condition diminished with time due to marine growth on the bottom surface of the specimens). In addition, during the winter months, the beams were subjected to cycles of freezing and thawing with each tide when the air temperature was at or below 28°F (freezing point of seawater). One cycle of freezing and thawing was completed each time the temperature of the center of the beam passed below 28°F and then above 28°F. The number of freeze/thaw cycles has ranged from 85 to 242 per year with an average of 135 cycles per year over the entire exposure period.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

Inspection of specimens

3. Inspection of the beams was conducted by the resident inspector weekly. At these inspections, adjustments were made to the loading yokes to maintain the gap openings of the spacer gages. This was continued until 1959. In addition to this weekly maintenance the specimens were inpsected annually by a team of observers to evaluate the deterioration of the beams. Two methods of rating were used.

4. From 1951 to 1959 the beams were evaluated according to the following system:

Condition	Score	Numerical Rating
Negligible deterioration	1	100
Slight deterioration	2	75
More advanced deterioration	3	50
Advanced deterioration, usually with considerable exposure of reinforcing steel	4	25
Complete loss of load-carrying capacity	5	0

When an evaluation by one observer departed appreciably from those of the other observers, his evaluation was not considered; however the opinions of the observers were remarkably concordant with very few discrepancies. Since 1959, a new evaluation system, devised by Mr. R. L. Bloor, then of OCE, has been used; the scoring in this system is essentially as shown below:

Condition	Score	Numerical Rating
Negligible deterioration	0	100
Slight deterioration	4	75
More advanced deterioration	104	50
Advanced deterioration, usually with considerable exposure of reinforcing steel	129	25
Disintegrated, incapable of carrying load	629	0

5. The method of scoring was adjusted to bring the tensile crack study in line with other studies taking place at the time. The score given to each beam is a function of the instructions given the raters.

For reference, this set of instructions has been included in Appendix A. The rating given each beam for the year is the numerical average of the 10 ratings awarded by the raters.

6. Other measurements were made annually on this series of beams. Maximum crack widths were measured annually since 1956 (except in 1959), and since 1953, pulse velocity tests have been made on all sound specimens.

7. By the end of January 1956, all of the nonair-entrained beams in Series A had failed, and testing and observation were concluded on all but 18 beams in the series (Nos. 1-18). From 1956 until December 1975, the previously mentioned tests were continued on the remaining beams.

Reports

8. Two interim reports[1,2] describing in detail the background information paraphrased here have been written. The first report describes tests and results up to July 1955, the second reports on activities between the years 1955 and 1963 and gives detailed information on the tests and observations to July 1963.

9. Subsequent to the July 1975 inspection tour, there were 13 Series A beams remaining at the exposure station. Eleven of these beams, still in testable condition, were returned to the WES in December of 1975 for testing and conclusion of the Series A Tensile Crack Program.

Purpose

10. The primary purpose of this phase of the investigation is to gather data from the annual observations and laboratory testing of the remaining beams in the series and to correlate this information with the parameters under study at the outset of the investigation. Of interest are the parameters of amount of load on the beam, the effect of exposure position of the beam, corrosion loss of cross-sectional area, casting position of the reinforcing steel, width and position of crack with relation to amount of corrosion on the reinforcement, chloride content, and depth of carbonation penetration of the concrete.

Scope

11. The work done under this phase of the investigation included return of the eleven beams to the testing laboratory, and visual examination and testing of the concrete and reinforcing materials. The tests conducted on Beams Nos. 1, 2, 4, 5, 6, 7, 8, 10, 11, 12, and 18 were as follows:

- a. Visual examination of the beams, including photographic recording of rusting, spalling, and staining.
- b. Examination and cataloging of the rust on the steel reinforcement to determine extent of corrosion.
- c. Structural testing of selected reinforcing bars to determine tensile strength, and the stress-strain characteristics of the steel.
- d. Tests for degree of chloride contamination.
- e. Tests for depth of carbonation.

PART II: MATERIALS AND MIXTURES

Cement and Air-Entraining Admixture

12. The cement used for all 82 beams was a type II portland cement that met the requirements specified in CRD-C 200[3] for type II cements. It was designated RC-220. The air-entraining admixture used was a commercially prepared neutralized vinsol resin. The physical and chemical properties of the cement are presented below:

Chemical Properties, %	
SiO_2	22.5
Al_2O_3	4.8
Fe_2O_3	3.4
CaO	63.0
MgO	3.1
SO_3	1.9
Na_2O	0.14
K_2O	0.50
Total Na_2O and K_2O reported as Na_2O	0.47
C_3S	43.0
C_2S	32.0
C_3A	7.0
C_4AF	10.0
$CaSO_4$	3.2
Loss on ignition	0.71
Insoluble residue	0.56

Physical Properties	
Fineness, Wagner	1755 sq cm/g
Fineness, Blaine	3255 sq cm/g
Heat of hydration by heat of solution method:	
7 days	76 cal/g
28 days	87 cal/g

(Continued)

Physical Properties	
Time of set, initial	2 hr 29 min
(Gillmore), final	4 hr 55 min
Autoclave expansion	0.12%
Air content of mortar	4.50%
Compressive strength of mortar:	
3 days	1815 psi
7 days	2960 psi
28 days	4860 psi

Aggregates

13. Manufactured limestone sand and crushed limestone coarse aggregate from a commercial source near Nashville, Tennessee, were used in the concrete. The limestone was approximately 60 percent oolitic fossiliferous, 20 percent porous or weathered argillaceous, 15 percent cherty argillaceous, and 5 percent other material. Physical properties and test results follow:

	Coarse Aggregate	Fine Aggregate
Specific gravity	2.71	2.69
Absorption	0.4%	1.2%
Loss in 5 cycles $MgSO_4$	2.9%	7.4%
Loss in Los Angeles abrasion test	25.4%	--
Relative compressive strength mortar cubes: 3 days	--	165%
7 days	--	141%

Average Grading (Three Tests)

Coarse Sieve	Coarse Cumulative % Passing	Fine Sieve	Fine Cumulative % Passing
1-in.	100	No. 4	100
3/4-in.	99	No. 8	92
1/2-in.	64	No. 16	71
3/8-in.	30	No. 30	44
No. 4	2	No. 50	25
		No. 100	12
		Fineness modulus	2.56

Concrete

14. The concrete was mixed in the laboratory in a Koehring rocking-tilting mixer. A separate batch was made for each reinforced beam. Each batch was tested for slump, and for air content if the concrete contained the air-entraining admixture. Six 6- by 12-in. cylinders were cast from the concrete representing each beam tested at 7 and 28 days. The chord modulus of elasticity, between 250 and 1000 psi, was determined on the cylinders tested at 28 days age. Test data concerning the concrete mixtures and average compressive strengths and modulus of elasticity are summarized below:

	Nonair-Entrained	Air-Entrained
Slump, in.	3 to 3-1/2	3 to 3-1/2
Air content, %	--	4.5 ± 0.5
Cement factor, bags/cu yd	5.20	5.35
Sand: Total aggregate, %	48.5	42
Water-cement ratio, by wt	0.70	0.60
Avg compressive strength, 7 days, psi	2650	2695
Avg compressive strength, 28 days, psi	3855	3820
$E \times 10^{-6}$, psi	4.94	4.86

Reinforcing steel

15. The reinforcing steel conformed to ASTM Designation A 16-50T[4] for "Rail-Steel Bars for Concrete Reinforcement," or to Designation A 15-50T[4] for "Billet-Steel Bars for Concrete Reinforcement," intermediate grade. The billet-steel bars conformed to ASTM Designation A 305-50T[4] for "Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement." Some of the rail-steel bars had deformations conforming to ASTM Designation A 305-50T[4] and the others had old-style deformations that did not meet the requirements of A 305. All working stresses were well below the yield strength.

Adaptation of Study Parameters

16. At the installation of the beams in 1951 the variables under study were: type of concrete (air-entrained versus nonair-entrained),

thickness of concrete cover over reinforcing steel (3/4 in. versus 2 in.), type of reinforcing steel (rail steel versus billet steel), type of deformations of the reinforcing steel (ASTM Standard A 305-50T[4] deformations versus old-style deformations), degree of tensile stress in reinforcing steel (0, 20,000, 30,000, 40,000, and 50,000 psi), and position of steel in the forms at the time of casting (top placed steel versus bottom).

17. Since the start of exposure, some of the variables have been eliminated because of failure of the specimens and lack of parameters for comparison. Sixty of the eighty-two beams in this series were nonair-entrained beams and all of these beams deteriorated to a point of total failure within five years after initiation of testing. The nonair-entrained beams contained all the billet-steel reinforcing bars, all the bars that were protected by 2 in. of cover, ~~and all the reinforcing bars~~ that had old-style deformations; consequently, all these variables were lost when the nonair-entrained beams failed. The variables remaining from the initial investigation which can be compared are the degree of stress in the reinforcing steel, the amount of corrosion to the bar, and the effect of casting the reinforcement in the top or bottom of the beam.

18. Two variable parameters have since been added to the evaluation of the beam exposure tests. These are the effect of the exposure position of the beam, and relation of crack width to amount of corrosion on the steel. Also the loss of cross-sectional area of steel, chloride content determination, and carbonation penetration tests were added to the testing program.

Specimens

19. All beams were 7 ft 9 in. long. The width of the beams varied between 8 and 10 in. Each beam contained two steel bars that were cast in either the top or bottom of the beam. Beams 1-46 had a 3/4-in. nominal cover over the reinforcement and beams 47-82 had a 2-in. nonimal cover. A summary of the loading, beam composition, and type and placement of steel for each beam is given in Table 1.

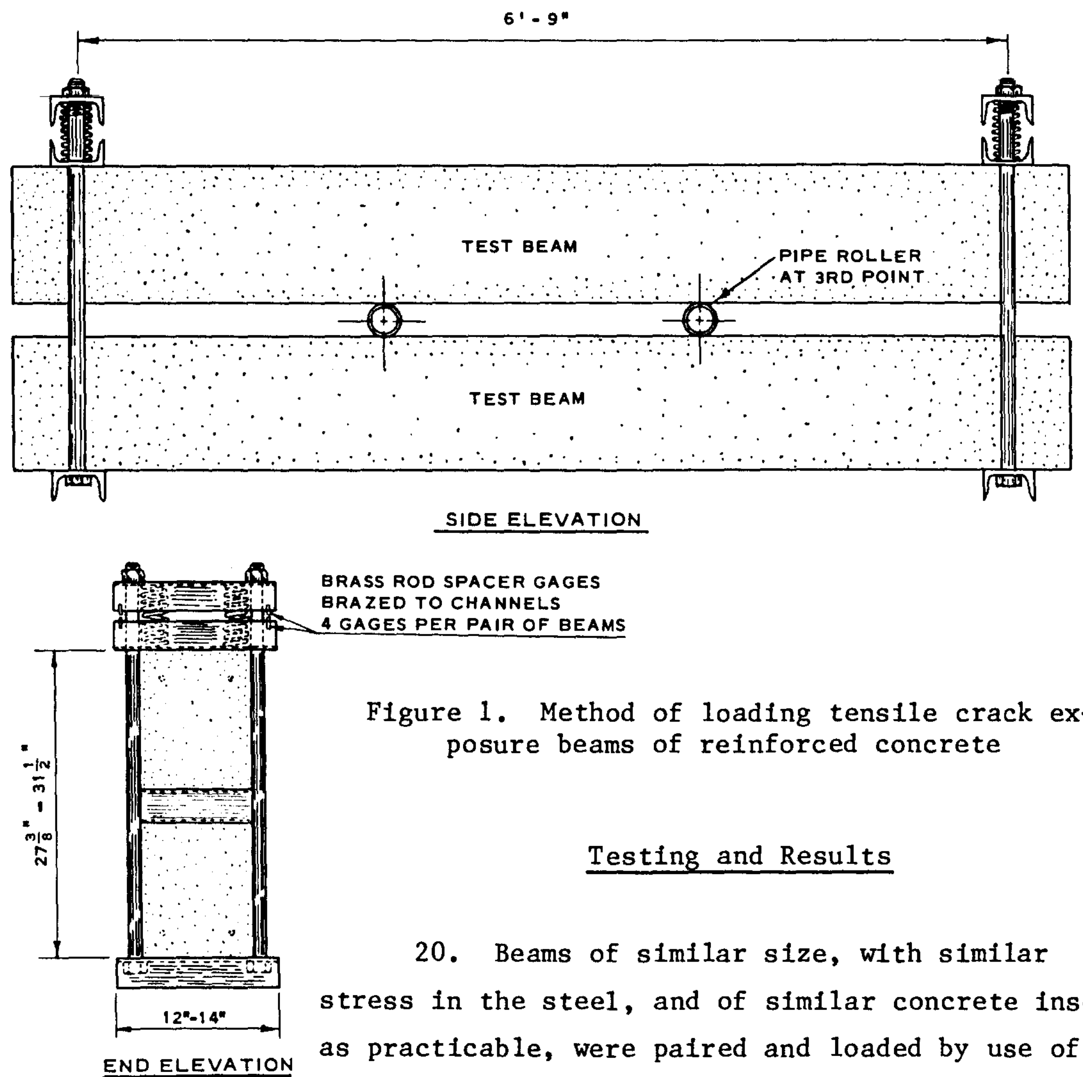

Figure 1. Method of loading tensile crack exposure beams of reinforced concrete

Testing and Results

20. Beams of similar size, with similar stress in the steel, and of similar concrete insofar as practicable, were paired and loaded by use of the spring and yoke devices shown in Figure 1. The

springs were placed between the two upper channel sections and loaded the required amount in a testing machine at which time the spacer-gage rods were set with a gap of 0.04 in. The rollers were placed at the third points, the yokes with springs were positioned, and the nuts on the yoke rods tightened the required amount to close the spacer gages to the original 0.04 in. The test beams were installed on concrete sills at mean-tide elevation on the beach at Treat Island. The ages of these specimens at the time of installation ranged from 90 to 120 days. The Series A beams were installed in November 1951. The beams were installed initially in an upright position on the beach, i.e. one beam of a pair was directly over the other beam of the pair. In the fall of 1956, the beams were turned on their sides to eliminate unequal exposure conditions ~~resulting from the upright~~ position.[1]

21. Cracks developed in all of the loaded beams during loading. Typical cracking is shown in Figure 2. The cracks were all fine and irregular, and the width of these cracks was not measured prior to

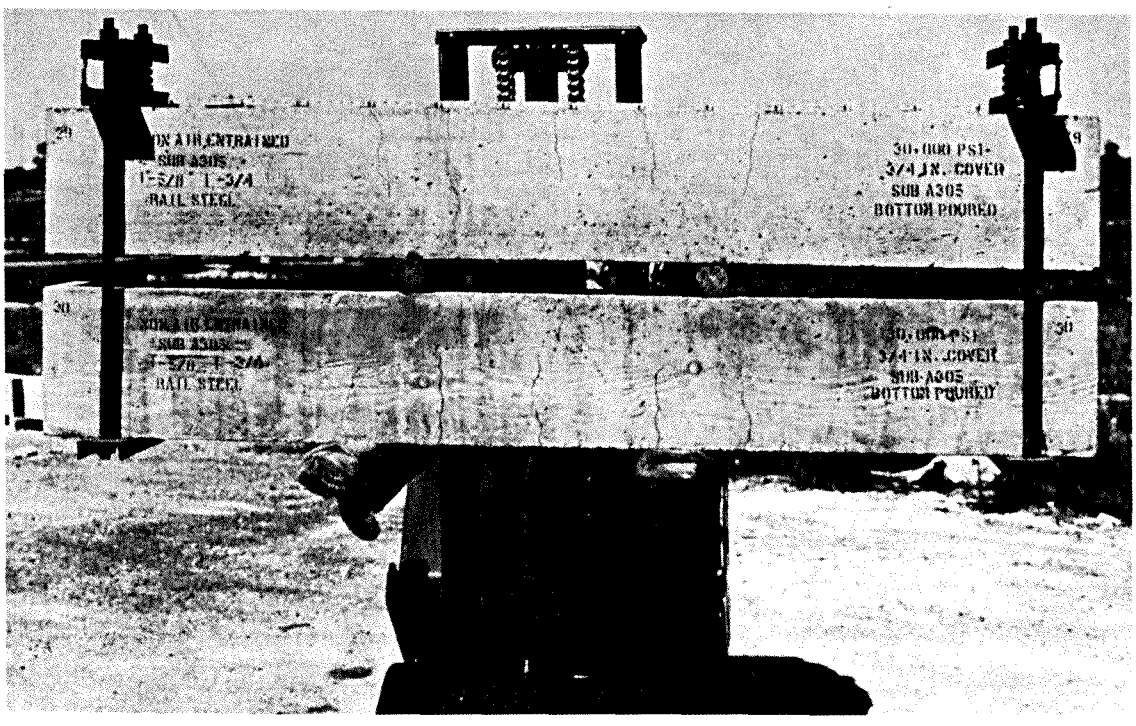

Figure 2. Typical cracking in loaded beams

exposure. Since 1956, the width of the cracks in both series of beams has been measured annually (except in 1959).

Laboratory Test Equipment

22. The beams returned to the WES for study were tested to failure in the apparatus shown in Figure 3. They were loaded in third-point flexural loading similar to that used during the exposure period. The ram is a 60-ton hydraulic ram supplied by an electrical hydraulic pump. Selected sections of the reinforcing bars were tensioned tested to failure in the 440,000-lb universal testing machine.

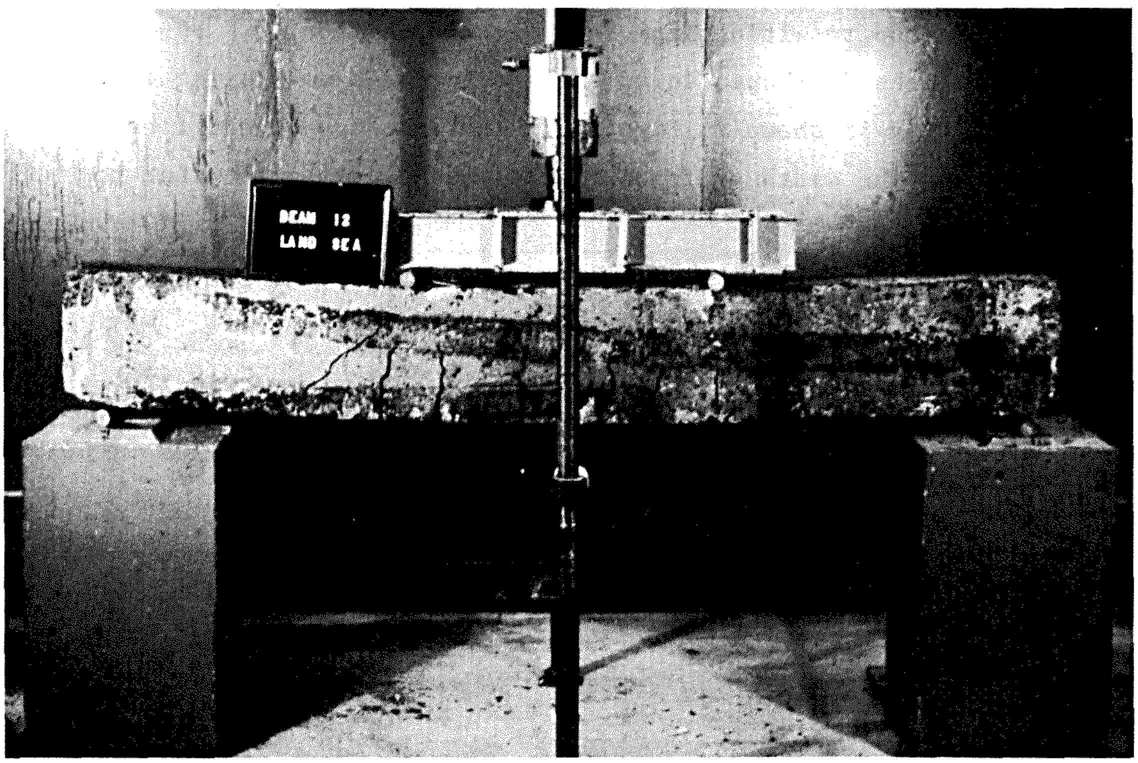

Figure 3. Failure testing configuration

PART IV: LABORATORY TESTS AND RESULTS

23. In order to protect the beams in transportation from the exposure site to the WES, they were wrapped in a protective bituminous membrane that prevented drying of the specimen. They were also placed on pallets to help protect them from damage while traveling.

24. When the beams were received in the laboratory and ready for testing they were removed from the protective membrane and set up for photographing the "as received" condition. Before photographs were made the beams were examined with a magnifying glass to determine the location of the flexural cracks that had been initiated by the loading placed on the beams at the start of the investigation. The cracks were painted so that they would be visible in the photographs and a record of the distance of each crack from the end of the beam was made.

Flexural Failure Tests

25. Seven of the eleven beams were tested to failure in third-point flexural loading. The beams were marked at supports and third points of the 85-in. test span, centered in the testing frame, and checked for longitudinal alignment and levelness. Two beams from the 20,000-psi range (Nos. 2 and 4), three beams from the 30,000-psi range (Nos. 5, 7, and 8) and one beam from the 40,000-psi range (No. 12) as well as beam No. 18, which was exposed without being stressed, were flexurally loaded to failure at a constant loading rate of 2000 lb per minute (Figure 3). The midspan deflection was measured by dial gage with least reading of 0.001 in. at every 2000-lb increment of load until failure. At failure, the ultimate load and deflection were recorded (see Table 2) and the beams were photographed to record the failure condition.

26. The beams were examined to determine the amount of corrosion to the reinforcing steel. When they were received in the laboratory, sketches were made to show the extent of spalling of the reinforcement cover. The length of the concrete spall, the length of exposed steel,

and the distance from the end of the beam were recorded. A sketch of the flexural cracks was made and their dimensions and locations were recorded. The cover over the steel was then broken away and the steel removed from the beam. Sketches of the corrosion to the steel were recorded to be matched to the crack locations of the concrete (Plates 1-6).

Depth of Carbonation

27. After the steel had been removed from the beams, the remaining concrete was prepared for carbonation examination. Two slices of concrete, approximately 3/4 in. thick, were sawn from each end of the beams and the middle. The slices were 8-10 in. wide and extended from the top surface of the beam down to the level of the steel. They were painted with a phenolphthalein indicator and examined to determine the depth from the surface that the concrete was carbonated (Photos 1 and 2).

Chloride Content

28. Each beam was sampled to determine the chloride content in relation to the depth from the surface of the beam. Samples were taken from the cross section at two levels. One was taken just above the level of the steel to determine the chloride content in the vicinity of the steel and the other level was taken 4 in. from the top surface of the beam. Figure 4 shows the orientation of the chloride samples. The samples were obtained by drilling a 3/4-in.-diam hole in the cross section and collecting the concrete powder.

29. Chloride samples were taken from a cross section 8 in. from the landward end, from a cross section at the middle of the beam, and from one 8 in. from the seaward end for the first beam to be analyzed. The results of the chloride examination for that beam revealed that there was no appreciable difference in the concentration of chlorides from one end of the beam to the other; therefore, on subsequent beams the chloride samples were taken from a cross section at the center of

the beam. The results of the chloride concentrations versus depth from the surface of the beam are shown in Plates 7-11.

Ultimate Tensile Strength Tests

30. Selected portions of the reinforcement from the beams were tension tested to ultimate load to determine the ultimate strength, and the stress-strain properties of the bars. Three-foot-long sections of reinforcing bars were selected from the least corroded area of each bar to eliminate the factor of reduction in strength due to corrosion from the comparison of tensile properties of the steel with stress level. One No. 6 bar each from the 0, 20,000, and 30,000-psi stress level and one No. 5 bar from the 40,000-psi stress level was chosen to provide specimens that were in approximately the same condition and nearly the same diameter. The results of the structural testing were compared with ASTM Designation A16-57T[4] for rail steel bars used as concrete reinforcement. Structural testing results are given in Table 2.

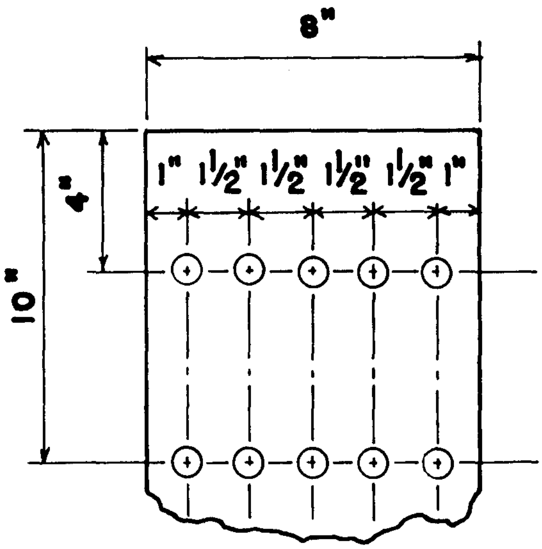

Figure 4. Orientation of chloride content samples

PART V: DISCUSSION OF RESULTS

31. Photographs 3-22 show the "as received" condition of ten of the beams. The surfaces of the beams were still moist with sea-water when they were taken from the bituminous membrane, and they had not been subjected to any drying in transit. As can be seen in the photographs there was considerable spalling of the cover over the reinforcement. The spalled areas range in length from 1-1/2 in. long, where no reinforcement had been exposed to nearly the entire length of the beam, exposing the major amount of reinforcement. The spalls were deep enough to expose at least half of the bar and generally occurred in the area of the flexural cracks, although some spalls did occur at the ends of the beam away from the flexural areas.

32. The flexural cracks initiated at time of loading in 1951 were all within the constant moment ~~section of the loading configuration and~~ their depth was related to the amount of load applied to the pair of yolked beams. The crack spacing between adjacent cracks varied from 3 to 18 in. with the majority of the cracks separated by 5 to 7 in. The spalled areas of the beams did not tend to span from one crack to the next but often crossed two or more flexural cracks and terminated somewhere between the cracked sections.

33. Longitudinal cracks were observed on several beams at the level of the reinforcement. They ran parallel to the reinforcement either on the sides of the beam or the bottom (Photos 23 and 24) extending from either a spalled area or a flexural crack.

34. The steel that was exposed at the spalled areas was heavily corroded. The corrosion was deep in some instances removing the deformations from the bar, and in others it just covered the surface of the bar. The corrosion covered the entire area of bar that was exposed by spalling. Some of the larger spalled areas exposed the entire surface of the bar and at smaller areas only part of the bar was exposed.

Flexure Tests

35. Beams 2, 4, 5, 7, 8, 12, and 18, representing stress levels

of 20,000 T,* 20,000 B,* 30,000 T, 30,000 B, 40,000 B, and unstressed, respectively, were loaded in third-point flexure to failure. The ultimate load and deflection for these beams are shown in Table 2.

36. Four of these seven beams, Nos. 2, 4, 5, and 7, arrived at the WES in the condition shown in Photo 25. They were broken at the center of their span. During transport to the WES they had been lifted by chain at the center span with the steel in the bottom half of the beam. This placed the unreinforced concrete at the top of the beam in tension and the concrete at the bottom of the beam in compression (note the crushing of the concrete at the bottom of the photograph) and the weight of the beam was enough to cause failure of the section.

37. These beams were tested in third-point flexure in the same manner as the unbroken beams. Although they were cracked all the way through their section, the method of loading placed part of the broken section back in compression and all the tensile stress on the reinforcement. This was similar to the stress conditions of the unbroken beams since they also had compression on the upper section and all tensile stress on the reinforcement (due to the deliberately cracked section of the test program). Whether or not the beams were broken when tested to failure, the ultimate load ranged from 28,600 to 39,800 lb (Table 2).

38. The mode of failure for the seven beams was diagonal tension. Six of the beam failures were initiated by pullout of the reinforcement from the concrete at the failure end (Photos 26 and 27). The other failure was also diagonal tension but it was not preceded by a bond failure (Photo 28).

39. There was no conclusive indication that the beams tested in flexure which had been broken in transportation from Treat Island experienced any different loading condition from those that arrived intact. Since the unbroken beams were already cracked by flexural loading during exposure, the concrete could not carry any tensile stress. Similarly, the broken beams, by being tested in the upright manner, experienced compression in the top fibers that were forced together by the method of

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* T means steel cast in top, B means steel cast in bottom.

loading and no tensile stress in the bottom fibers (all tension was taken by the steel). The beams all failed within an 11,200-lb range.

Ultimate Load Versus Stress Level

40. The data collected from the beams that were tested to ultimate failure are shown in Table 2. The ratio comparison given in the last column is the actual ultimate moment developed in the beam during flexural loading compared to the design ultimate moment. These ratios are significant where the failure loads are not because of the differences in beam cross section and amount of reinforcing steel at the different stress levels. The moment ratio developed at failure ranges from 0.839 in the control beam to 1.753 in the beam at the 40,000-psi stress level. The design ultimate moment was that calculated by ultimate strength requirements without a safety factor applied. The yield strength of the steel used in the calculations (shown in Appendix B) was 50,000 psi, which was the minimum specified yield strength for rail steel reinforcement at the time of casting. This assumption is fortified by the yield strength tests conducted during the laboratory testing period. The data in Table 2 show that with the exception of the control beam, all actual ultimate moments were greater than the design ultimate moments. The ratio of 1.753 of the beam at the 40,000-psi stress level is of doubt because its deflection at midspan is twice that of any other beam and the ultimate load recorded was probably a load recorded after failure. No definite relationship can be drawn from the data with respect to ultimate moment and stress level. However, it can be stated that the ultimate moments were not reduced below acceptable levels over the 24-year period due to stress, corrosion, or loss of bond length from spalling.

Casting Position of the Steel

41. One of the original purposes of this investigation was to determine the effect of position of the steel in the forms at the time of casting. When concrete is placed in a form, and vibrated, there is a

tendency for the heavier particles to migrate to the bottom of the form. Any object placed in the form that will not move forms a bridge that stops the downward migration of particles above the object and produces a pocket below the object that will contain more water and less cement. The higher up in the form that an object occurs, the greater the tendency for a weak pocket beneath the object to be formed. For this reason half of the beams at each stress level were cast inverted with the steel at the top of the form and the other half were cast normally with the steel at the bottom of the form.

42. Observations of the imprint of the reinforcing bar in the paste surrounding the steel showed that the paste above the bar was more dense, contained fewer air voids, and was harder to scratch. The areas of paste below the bars were chalky in texture, contained more air voids, and were less dense. Photo 29 shows a piece of concrete taken from one of the beams in which both the paste above and below the bar is exhibited. The paste in the top half of the photo is harder and more dense than that in the bottom half of the picture which as can be seen contains more voids. Another observation that was made was that regardless of whether the reinforcement was cast in the top or the bottom of the beam, the hardened paste did not segregate from the bar enough to destroy the pattern of the deformations in the paste. If the paste had segregated from the bar to a point where it no longer interlocked with the bar deformations, then in addition to lost bond the benefits of the deformations in transferring stresses from the concrete to the steel would also be lost. This phenomenon did not occur on any samples of concrete examined in this investigation.

Corrosion to the Reinforcement

43. Each beam was broken open to expose the reinforcement that was still protected by cover. In general, the steel beneath the 3/4-in. cover was not corroded. There were areas beneath the concrete that did receive corrosion. The tips of the reinforcing bars were generally rusted, and the areas of the bars where they were welded to the other

bar in the beam were generally corroded (Photo 30); but on the whole the bars that were not exposed to direct attack by oxygen and seawater remained only lightly rusted.

44. Photos 31 and 32 show the extent of corrosion directly adjacent to a spalled area. In both photographs, the darkened concrete area in the right of the pictures are areas where concrete had spalled away from the rest of the beam while at Treat Island, exposing the steel to water and oxygen. The lighter areas of each photograph show concrete and steel surfaces that were exposed in the laboratory by autopsy and reveal that the corrosion did not penetrate from the spalled area into the sound portion of the beam. Similarly, most of the steel areas that remained covered were unrusted. Photos 33-37 show areas of the steel from unspalled areas of various beams. The corrosion shown varies from moderately rusted (Photo 33) to unrusted (Photos 36 and 37) and is representative of the corrosion that appeared beneath the sound concrete cover.

45. Plates 1-6 are graphic representations of the beams in the "as received" condition. Each plate shows both faces of the beam and all areas of the reinforcement that were exposed. They also show the longitudinal and flexural cracks, spalled areas, and corrosion to the reinforcement. The sketch between the faces of the beams shows the condition of the reinforcement after it was removed from the concrete. The darkened areas on the bars represent the corroded areas. The plates show that the number and depth of flexural cracks increased with the increase in steel stress. They also show that the corrosion on the bars does not align with the flexural cracks in the concrete at lower steel stress levels. The corrosion on the bars that were stressed in the 20,000-psi stress range was, in general, at different locations than the flexural cracks. The corrosion at spalled areas was confined to the area of the steel directly exposed to water and oxygen; however, where there were flexural cracks and the concrete was not spalled, the bars beneath remained free from heavy corrosion (Photo 38). In the higher stress ranges, 30,000 and 40,000 psi, the corroded areas of the steel more generally matched the location of the flexural cracks (Plates 4 and 5);

however, at both stress levels there were areas where flexural cracks occurred and there was no corrosion and areas of steel that were corroded beneath uncracked concrete cover. Table 3 shows the maximum crack widths measured by stress levels; at the 20,000-psi stress level no corrosion was found at flexural cracks. In 1971 the smallest value of maximum crack width reported was 0.015 in. Therefore, from the exposure data it was found that no corrosion occurred at a crack width of less than 0.015 in.

46. In the majority of areas where there were longitudinal cracks in the beams, the steel beneath was heavily corroded. These areas were areas of the beginnings of future spalls, being caused by the pressure buildup between the reinforcement and the concrete by the corrosion products.

Stress Level Versus Crack Width

47. The data gathered by the teams of observers on the measurement of crack width over the period 1957 to 1975 are presented in Table 3. This information is graphed in Plate 12. The data show an increase in crack width with respect to both length of time under load and amount of stress applied to the reinforcement. In 1959 the original loading hardware was replaced with new hardware and the loads reapplied to the beams. Due to this reloading the 1960 crack width measurements are larger than either the 1958 or 1961 measurements. Stress relaxation and creep of the concrete caused the cracks to close between the years 1960 and 1961. The load on the beams was not reapplied again until 1967 and then it was reapplied every year until the end of the test period. The trend of crack width versus age is erratic until 1967 and then all the curves show an increase with respect to time. Yearly renewal of the stress in the reinforcement caused the increase in the crack width.

48. The data recorded during the erratic reloading procedures from the years 1957-1967 should be ignored in the evaluation of increase in crack width with respect to steel stress because there is no evidence that the actual stress in the steel was that specified at the outset of

the investigation. The data recorded from 1967-1975 are more reliable because the beams were reloaded annually to the initial steel stress.

49. The last year in which a complete comparison between stress level in the steel and crack width can be made on all four beams at each stress level is 1971. Table 4 is a continuation of a table presented in Report No. 2[2] of this series, relating the stress level in the reinforcement to the average of the maximum crack width for various years, and the change in maximum crack width from 1957 to 1971. These data show the average maximum crack width for beams at each of the stress levels for years 57, 63, 66, 69, and 71, and the average change in maximum crack width over the 14-year period. The change in maximum crack width increases with increase in stress in the reinforcing steel except for the 50,000-psi stress level. The beams at this level showed a smaller change in the maximum crack width for the years 1957-1971; however, inclusion of the data for the year 1972 into the average change of maximum crack width indicates that the maximum change occurs at the 50,000-psi stress level. The indication from the data is that the crack width increases with yearly reloading of the beams and that the maximum width is roughly proportional to the stress level in the beam. This later conclusion has been reported previously by Gergely and Lutz.[5]

Stress Level Versus Tensile Properties of Steel

50. The stress-strain properties of selected pieces of steel were recorded to determine if the stress level during exposure had any effect on the properties of the steel. Table 5 shows the results of tensile testing of four selected pieces of steel. The samples were taken from the reinforcement in areas of the bar where there was minimum corrosion. As can be seen from the table all bars tested exhibited greater than minimum specifications of ultimate tensile strength, and yield point according to ASTM Designation A16-57T.[4] The ultimate strength was calculated as the ultimate tensile load divided by the cross-sectional area of the bar, and the yield point was taken as that point in the stress-strain curve where the curve deviated from linearity of the elastic

range. The data show a decrease in the yield point of the steel with an increase in load; however, this is not considered sufficient to indicate a reduction in strength with increasing stress level during exposure since the trend is not reflected in the ultimate strength data.

51. The steel tensile properties did satisfy the minimum ASTM standards for rail steel after 24 years of severe exposure. It should be mentioned that the steel specimens were taken from areas of the bar where there was only mild surface rust and that the data presented do not represent areas of the steel that were directly exposed to the environment.

Stress Level Versus Steel Corrosion

52. The diameter of each bar was measured to determine the minimum cross-sectional area of the bar to try to find a relationship between the amount of corrosion and the stress level. Table 6 shows the minimum cross-sectional area and the percent reduction of area for each bar grouped at stress levels. The average reductions for the 20,000- and 30,000-psi stress ranges were 18.97 and 16.73 percent, respectively, while the average for the bars at the 40,000-psi level was 41.33 percent. This might appear to be an increase in percent reduced area with increasing stress; however, the control beam which was under 0-psi stress had an average reduction in cross-sectional area of 32.93 percent. From the data collected it seems that no relationship between stress level and maximum reduction in cross-section area can be made. The areas of maximum reduction in cross section occurred at spalled areas of the beam (Plates 1-6) in 15 out of 22 bars. Here the steel was directly exposed to salt water and oxygen. In the remaining seven bars the maximum reduction in cross-sectional area did not occur at flexural cracks in the concrete indicating that although corrosion did occur at cracks it was not the worst area in the beam.

Annual Condition Rating

53. Table 7 shows the annual numerical ratings given the beams

from the year 1951 through 1975. Each entry is the average of the ratings of four inspectors given in each year. The ratings reflect the condition of the exterior of the beams according to the rating systems described in paragraph 4. Plate 13 shows this data in graph form. The data show a continual drop in annual rating with respect to time. Individual data points showing increase in numerical rating with time are the result of different inspectors in different years. However, the overall effect was a continual decrease in condition with time ranging from negligible deterioration at time zero to advanced deterioration of all beams at 24 years.

54. The beams in all stress levels showed approximately the same amount of deterioration during the first eight years, and over a 20-year period from 1951 to 1972 when full evaluation of all the beams at each stress level could be made. All the beams except those in the 20,000-psi stress range exhibited similar characteristics of deterioration. The beams in the 20,000-psi stress range showed advanced deterioration much earlier than the rest of the beams. The order of deterioration by stress levels over the 20-year period from least to worst is: 0-, 40,000-, 50,000-, 30,000-, and 20,000-psi stress. Roshore[2] found that after 12 winters of exposure the order of durability was 0, 50,000, 40,000, 20,000, and 30,000 psi. His results were based on comparisons of the 1963 numerical ratings while the results shown here are a result of graphic evaluation of the data over the entire test period.

55. In both sets of results the beams in the 20,000-psi stress range proved to be the least durable. In the present data the beams in this stress level showed rapid deterioration between 1958 and 1959. This condition is not well understood; however, it may be explained by the fact that in May of 1959 the beams were reloaded to their initial stress levels. This reloading could have caused serious spalling, and crack widening of the beams at this stress level such that their numerical ratings would be heavily reduced. It is also theorized that the size of the reinforcing bars would add to this reduced durability. In the 20,000-psi stressed beams the steel was No. 7 and No. 6 bars (Table 1). This large diameter gives a large surface area for corrosion products to

attack. If there indeed was greater pressures on the concrete from rust depositions the reloading in 1959 may have been enough additional stress to cause large amounts of spalling in this one year. (Spalling was counted heavily in the evaluation reports.) The beams in the 30,000- through 50,000-psi range had bars that ranged from No. 6 to No. 4 and their reloading may not have produced sufficient stresses to spall the concrete.

Chloride Contamination

56. The amount of chlorides that penetrated the beams over the 24-year exposure period ranged from 0.12 to 0.70 percent by weight of concrete sample. Plates 7-11 show the distribution of the chloride concentrations across the cross section of the beam. In the exposure all the loaded beams were oriented with one side of each beam facing upward and one side facing downward. This was done to eliminate uneven exposures that resulted if the loaded pairs were positioned one on top of the other. The chloride content of the beams ranged from 4.68 to 27.32 lb/cu yd of concrete. These values indicate chloride levels great enough to cause corrosion in the steel. These plates show the general trend of concentrations of chloride. They were greatest closest to the surface of the beam and least at the center line of the beam. Each beam is represented by two graphs. One graph (Samples 6-10) represents samples taken 4 in. down from the compression face of the beam and the other (Samples 1-5) taken 1 in. above the plane of the reinforcement. Beams 1, 2, 4, 5, 6, 7, 8, 10, 11, and 12 were exposed laying on their side, producing one side of the beam that was constantly facing upwards and one side downwards. Beam 18, the control beam, was not loaded and it was exposed with its tension face resting on the beach and both sides equally exposed.

57. Of the beams resting on their sides, it is evident that one side contained higher concentrations of chlorides than the other. The side labeled top side (Plate 7) shows the higher concentration of chlorides in all but one case (Beam 12, Sample 10). This phenomenon

results from the exposure position of the beams. With the beams exposed on their sides, the bottom side acquired a cover of algae and seaweed that prevented drying during hours of low tide. This prevention of drying occurred only on the bottom side. The top side not covered by seaweed was exposed to sunlight and evaporation drying. Consequently, the pores in the concrete covered by the seaweed remained filled with water containing chlorides while the pores in the top side experienced evaporation of the water and deposition of the chlorides in the pores. At each subsequent high tide the top side pores absorbed additional salt water while the pores in the bottom side, already filled with water, could not take on additional salts. With each wetting and drying, the top side increased its chloride content while the bottom side only increased its chloride content when it could dry out its pores.

58. The two graphs of Beam 18 do not show this phenomenon. Their curves are the characteristic curves that represent lower concentrations at the center line of the beam and higher concentrations close to the surface. Also the concentrations near the surface are nearly equal. It can be pointed out that the concentrations in Samples 6-10 are higher than those in Samples 1-5, subscribing to the hypothesis that the faces subjected to greatest drying effect will be able to absorb greater concentrations of chlorides. This would be particularly important with respect to marine structures that would be wettted and dried cyclically.

Carbonation Penetration

59. Photographs 1 and 2 show cross sections of the beams after they had been treated with phenolphthalein. Phenolphthalein is an indicator that turns red in the presence of alkalinity greater than a pH of 8.2. These photographs show that the entire cross section has turned red indicating that the alkalinity of the concrete was higher than 8.2. There was some small amount of carbonation penetration at the surface of the beams that amounted to a small border around the periphery of the cross-sectional slabs remaining colorless, but this did not play a role in the contamination of the steel.

PART VI: CONCLUSIONS

60. It is concluded from the laboratory tests and the examination of the data collected annually on Beams 1-18 of the Series A tensile crack exposure studies that although some of the beams returned to the laboratory were broken in transport, the data on ultimate moment derived from flexural failure of these beams can be considered significant due to the nature of the third-point flexural loading. Furthermore the ratio of the actual ultimate moment to design ultimate moment does not show any definite relationship to level of stress in the beam, except that stressing the steel during the exposure period did not reduce the ultimate moment capacity of the beams below acceptable levels.

61. With respect to position of the steel in the forms at the time of casting it is concluded that the position of the bars in the top or bottom of the forms did not affect the ultimate load-carrying capacity of the beams.

62. The steel was found to be heavily corroded at spalled areas and generally free from corrosion beneath the 3/4-in. cover. Since corrosion could not be found at any cracks in the beams stressed at the 20,000-psi level it is concluded that crack widths greater than 0.015 in. were necessary to produce corrosion at flexural cracks.

63. The data on crack width recorded before 1967 is unreliable because the stress levels were not maintained yearly. From 1967 to the termination of exposure, crack widths increased with respect to both time and stress level in the steel and the resulting crack widths are roughly proportional to the stress level of the steel.

64. It was determined through testing that the various stress levels in the beams during the exposure years did not reduce the tensile properties of the steel below minimum acceptable standards, and there was no apparent relationship between the ultimate load or yield point of samples tested in the laboratory and the stress level to which they were subjected during the exposure years. It is therefore concluded that the steel was not affected by the magnitude of the stress level imposed on the steel during exposure.

65. The percent reduction in cross-sectional area of the steel due to corrosion did not exhibit any relationship to stress level during testing. The maximum areas of corrosion occurred at spalled areas and no areas of maximum reduction of cross-sectional area occurred at flexural cracks.

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2. Roshore, E. C., "Tensile Crack Exposure Tests; Results of Tests of Reinforced Concrete Beams, 1955-1963," Technical Memorandum No. 6-412, Report 2, November 1964, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

3. U. S. Army Engineer Waterways Experiment Station, CE, Handbook for Concrete and Cement, Aug 1949 (with quarterly supplements), Vicksburg, Miss.

4. American Society for Testing and Materials, Book of ASTM Standards, [issued in parts], Revisions issued annually, Philadelphia, Pa.

5. Gergely, P. and Lutz, L. A., "Maximum Crack Widths in Reinforced Concrete Flexural Members," American Concrete Institute, Special Publication SP-20, Paper No. 6, 1968, pp 87-118.

Table 1

Description of Specimens Cast in 1951 (Series A)

No. of Beams	Beam No.	Beam Dimension in. Width	Beam Dimension in. Depth	Nominal Steel Stress psi	Applied Load at Each End, lb	Reinforcing Bar No.	Reinforcing Bar Size	Reinforcing Bar Deformation
			Air-Entrained Concrete, 3/4-in. Cover, Top-Positioned, Rail Steel					
2	1, 2	8	12-3/16	20,000	7500	1 1	No. 6 No. 7	A 305
2	5, 6	8	12-5/8	30,000	8600	1 1	No. 5 No. 6	A 305
2	9, 10	9	12-9/16	40,000	8000	1 1	No. 4 No. 5	A 305
2	13, 14	10	13	50,000	8150	2	No. 4	A 305
			Air-Entrained Concrete, 3/4-in. Cover, Bottom-Positioned, Rail Steel					
2	3, 4	8	12-3/16	20,000	7500	1 1	No. 6 No. 7	A 305
2	7, 8	8	12-5/8	30,000	8600	1 1	No. 5 No. 6	A 305
2	11, 12	9	12-3/16	40,000	8000	1 1	No. 4 No. 5	A 305
2	15, 16	10	13	50,000	8150	2	No. 4	A 305
2	17, 18	8	12-3/16	None	--	1 1	No. 6 No. 7	A 305

Table 2

Ultimate Load Properties of Tensile Crack Beams Tested in Flexure

Beam No.	Stress Level psi	Stress Position	Failure Load lb	Midspan Deflection in.	Moment Developed At Ultimate Load ft-kips	Design Ultimate Moment ft-kips	Ratio $\frac{M'_u \text{ Actual}}{M'_u \text{ Design}}$
2	20,000	Top	39,800	0.440	46.98	43.33	1.084
4	20,000	Bottom	38,000	0.517	44.84	43.33	1.035
5	30,000	Top	34,700	0.790	40.95	33.68	1.216
7	30,000	Bottom	28,600	0.800	33.75	33.68	1.002
8	30,000	Bottom	37,800	0.490	44.60	33.68	1.324
12	40,000	Bottom	33,750	1.606	39.82	22.71	1.753
18	Unstressed	Bottom	30,800	0.240	36.34	43.33	0.839

Table 3

Maximum Crack Widths, Series A

Beam No.	Nominal Steel Stress psi	Maximum Crack Width, 10^{-3} in. 1957	1958	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975
1	20,000	15	15	25	15	15	10	10	10	10	10/10	10	15	15	20	10*	**	**	**
2	20,000	5	5	15	10	15	10	10	5	5	5/5	10	10	15	15	**	**	**	**
3	20,000	10	10	15	10	10	10	5	10	10	10/15	20	20	20	25	**	**	†	†
4	20,000	5	5	10	10	5	10	10	10	5	5/10	20	25	30	30	15*	**	**	10††
5	30,000	10	10	25	15	10	20	15	20	10	10/15	20	20	25	25	30	30	25	30
6	30,000	10	10	20	10	10	10	10	10	5	5/5	15	20	25	30	30	10	15	30
7	30,000	10	10	25	15	10	15	10	10	10	10/10	20	25	30	35	25	20	20	30
8	30,000	5	5	25	15	15	20	15	10	10	10/10	20	25	30	35	40	30	30	30
9	40,000	10	10	35	15	15	20	20	20	25	25/25	25	30	35	45	30	25	†	†
10	40.000	15	15	25	15	30	40	30	25	30	25/35	40	45	50	55	50	50	††	††
11	40,000	10	10	25	10	20	20	20	25	20	20/30	30	35	35	40	35	40	40	45
12	40,000	15	15	35	15	15	20	20	20	20	15/30	30	35	40	40	50	50	50	50
13	50,000	15	15	60	15	10	20	25	30	30	25/35	35	35	35	40	50	60	†	†
14	50,000	20	20	30	15	15	20	20	20	20	20/25	25	30	35	40	60	50	††	15††
15	50,000	20	20	30	20	30	40	40	30	25	25/25	30	30	35	40	50	‡	†,‡	†,‡
16	50,000	10	10	20	10	10	20	15	25	25	20/25	30	35	35	40	40	††	††	‡
17	None	0	0	Exposure discontinued in 1960															
18	None	0	0	0	0	0	0	0	0	0	0/0	0	0	0	0	0	0	0	0

Note: Tests had been concluded on all other Series A beams in 1957. Crack measurements were not made on Series A beams in 1959. Dual readings in 1967 indicate crack widths before and after reloading beams.

* Beam spalled at cracked area and small crack visible.

** Beam spalled at cracked area, no crack visible.

† Returned to laboratory.

†† Not loaded.

‡ Failed.

Table 4

Average Maximum Crack Widths and Change, 10^{-3} in.

Stress Level psi	1957	1963	1966	1969	1971	1957-1971 Change
0	0	0	0	0	0	+0.00
20,000	8.75	10.00	7.50	17.50	22.50	+13.75
30,000	8.75	16.25	8.75	22.50	31.25	+22.50
40,000	12.50	25.00	23.75	36.25	45.50	+33.00
50,000	16.25	25.00	25.00	32.50	40.00	+23.75

Table 5

Tensile Properties of Selected Samples of Steel

Beam No.	Steel Stress psi	Measured Yield Strength psi	ASTM Minimum Yield Strength psi	Measured Ultimate Strength psi	ASTM Minimum Ultimate Strength psi
18	0	60,096	50,000	119,772	80,000
2	20,000	56,218	50,000	111,590	80,000
5	30,000	53,311	50,000	95,000	80,000
12	40,000	53,757	50,000	117,500	80,000

Table 6

Minimum Cross-Sectional Area and Percent Reduction

Stress Range	Beam No.	Bar Size	Measured Diameter, in.	Minimum Cross-Sectional Area, $in.^2$	Reduction of Area, percent
20,000	1	6	0.722	0.4094	6.95
		7	0.782	0.4803	19.95
	2	6	0.636	0.3177	27.80
		7	0.778	0.4754	20.77
	4	6	0.697	0.3816	13.28
		7	0.762	0.4560	23.99
				Average	18.97
30,000	5	5	0.601	0.2837	8.49
		6	0.722	0.4094	6.95
	6	5	0.600	0.2827	8.79
		6	0.703	0.3882	11.79
	7	5	0.600	0.2827	8.79
		6	0.622	0.3038	30.94
	8	5	0.558	0.2445	21.12
		6	0.594	0.2771	37.02
				Average	16.73
40,000	10	4	0.487	0.1863	6.86
		5	0.428	0.1438	53.59
	11	4	0.000*	0.0000	100.00
		5	0.540	0.2290	26.12
	12	4	0.452	0.1606	19.77
		5	0.480	0.1809	41.63
				Average	41.33
0	18	6	0.520	0.2123	51.74
		7	0.810	0.5153	14.12
				Average	32.93

* Bar was broken through when removed from beam.

Table 7

Numerical Ratings of Series A Beams

Beam No.	Normal Steel Stress psi	Yearly Rating 1951 - 1975 Year 51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75
1	20,000	100	93	93	83	76	64	76	70	26	26	29	27	29	26	*	24	*	24	29	29	26	28	24	26	23
2	20,000	100	96	91	82	91	77	71	65	55	55	48	50	46	44		39		39	46	46	29	42	17	31	20
3	20,000	100	96	95	89	78	69	73	79	26	26	32	30	39	30		28		28	28	31	28	28	29	**	**
4	20,000	100	93	85	84	91	76	64	72	46	46	44	44	46	44		42		37	37	38	16	30	30	†	†
5	30,000	100	93	98	95	92	77	79	80	63	63	61	60	57	54		50		50	54	54	54	54	52	52	51
6	30,000	100	93	89	91	89	84	80	78	73	73	63	60	71	60		61		57	56	57	52	47	91	44	40
7	30,000	100	96	100	91	79	47	32	35	25	25	34	32	32	31		31		31	33	33	33	32	31	33	30
8	30,000	100	96	95	91	94	82	76	71	75	75	62	60	59	63		62		55	55	61	48	50	44	50	30
9	40,000	100	89	85	82	94	78	77	75	66	66	64	63	60	63		62		56	59	59	50	22	21	**	**
10	40,000	100	93	100	92	89	76	81	79	67	67	66	63	67	66		60		47	50	62	22	22	72	†	†
11	40,000	100	89	85	82	91	70	64	57	51	51	50	50	51	50		46		46	47	49	45	42	46	43	41
12	40,000	100	93	98	94	89	80	80	81	75	75	61	59	61	59		55		50	51	58	45	51	46	49	45
13	50,000	100	96	100	95	86	78	74	81	59	59	51	48	48	51		34		28	22	19	18	17	16	**	**
14	50,000	100	93	88	80	89	80	65	70	60	60	60	58	60	60		58		58	58	58	55	58	50	†	†
15	50,000	100	93	95	89	86	85	78	69	59	59	57	57	56	58		56		54	51	56	50	34	††	††	††
16	50,000	100	93	83	82	92	86	76	84	57	57	55	56	55	55		55		53	54	52	53	50	†	†	†
17	0	100	96	90	87	89	82	71	74	66	‡															
18	0	100	96	100	95	100	90	79	73	67	67	56	54	65	55		60		55	55	53	55	55			

* In 1965 and 1967, condition of the specimens was not rated by a panel of observers.
** Returned to laboratory.
† Damaged, unloaded.
†† Failed.
‡ This beam was removed from exposure in January 1960 for testing, and not returned.

Photo 1. Section of 10-in.-wide beam treated for depth of carbonation

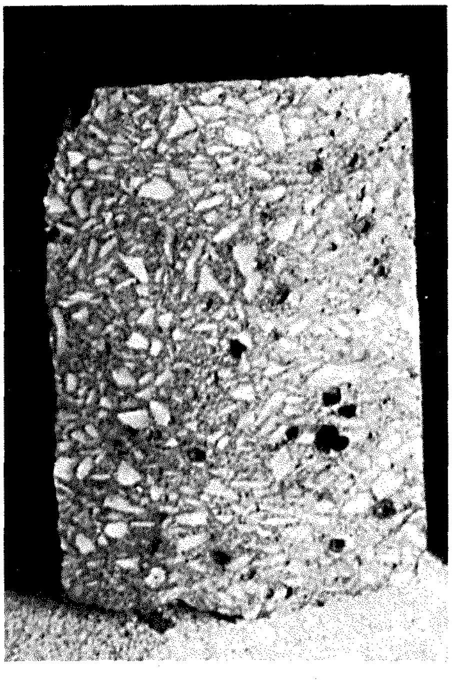


Photo 2. Section of 8-in.-wide beam treated for depth of carbonation

Note: In titles of Photos 3-22 the direction (east and west) refers to orientation at exposure station.

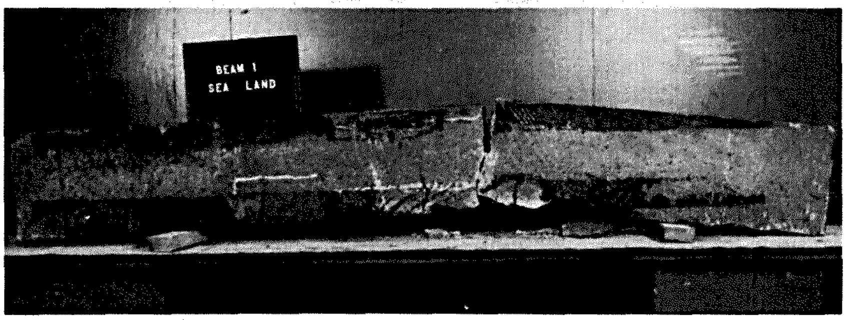


Photo 3. "As received" condition of beam 1 looking west

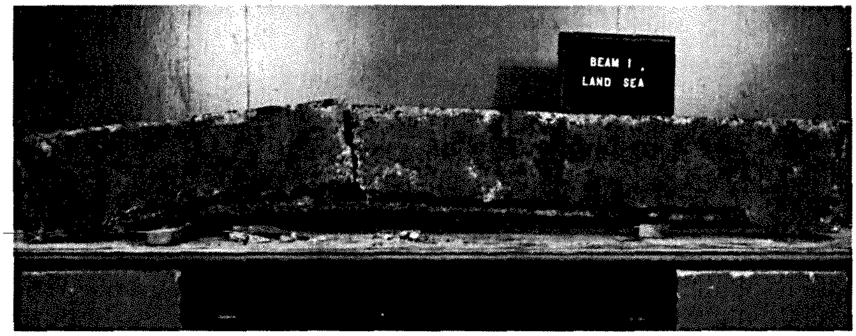


Photo 4. "As received" condition of beam 1 looking east

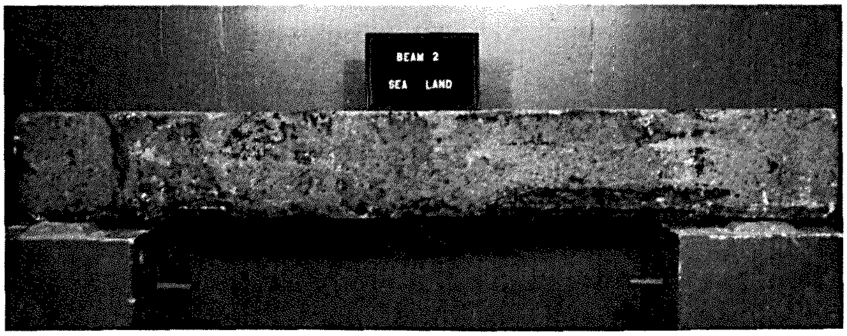


Photo 5. "As received" condition of beam 2 looking west

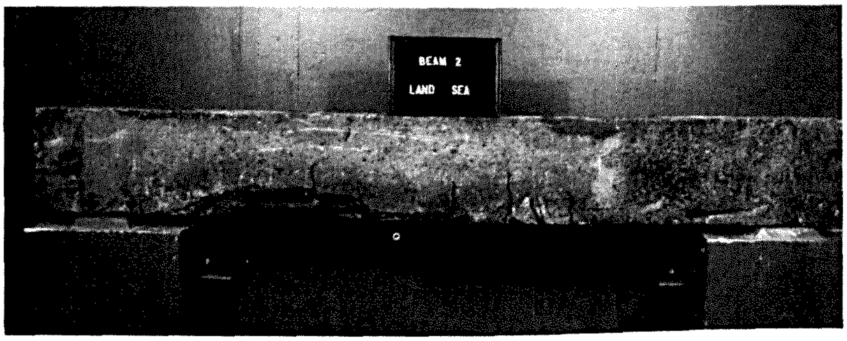


Photo 6. "As received" condition of beam 2 looking east

Photo 7. "As received" condition of beam 5 looking west

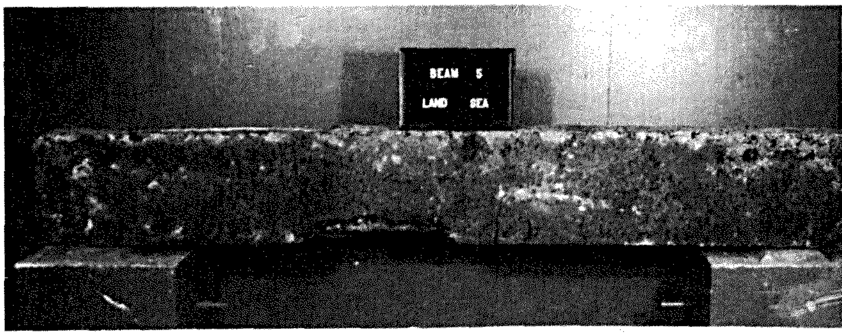


Photo 8. "As received" condition of beam 5 looking east

Photo 9. "As received" condition of beam 6 looking west

Photo 10. "As received" condition of beam 6 looking east

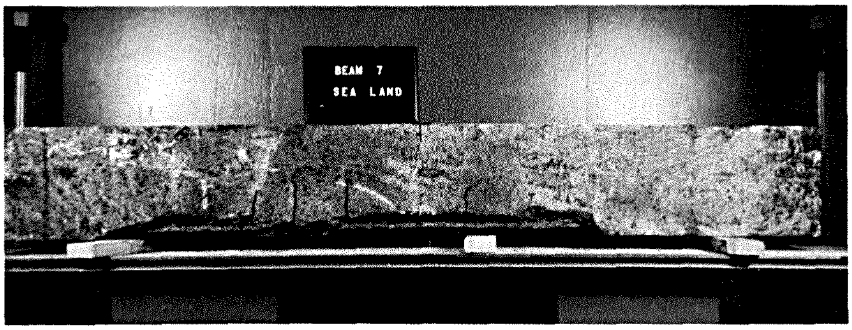


Photo 11. "As received" condition of beam 7 looking west

Photo 12. "As received" condition of beam 7 looking east

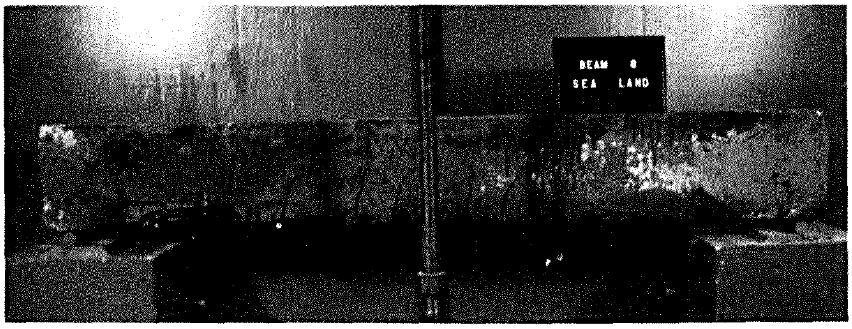


Photo 13. "As received" condition of beam 8 looking west

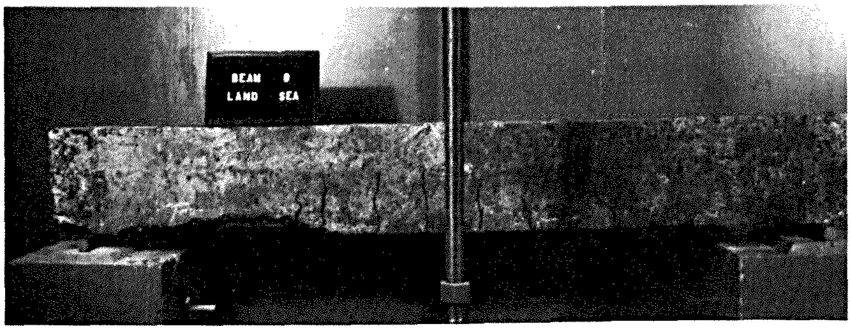


Photo 14. "As received" condition of beam 8 looking east

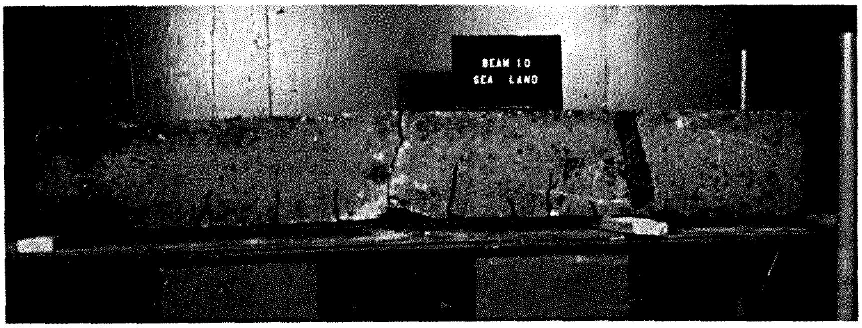


Photo 15. "As received" condition of beam 10 looking west

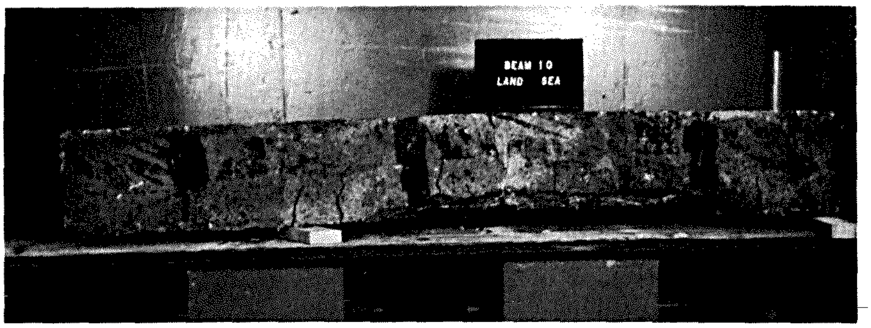


Photo 16. "As received" condition of beam 10 looking east

Photo 17. "As received" condition of beam 11 looking west

Photo 18. "As received" condition of beam 11 looking east

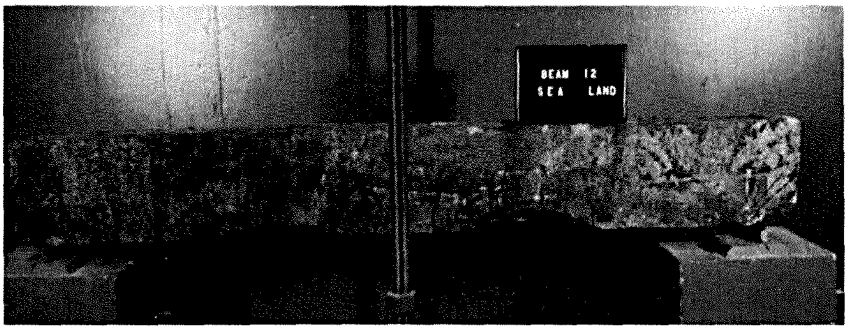


Photo 19. "As received" condition of beam 12 looking west

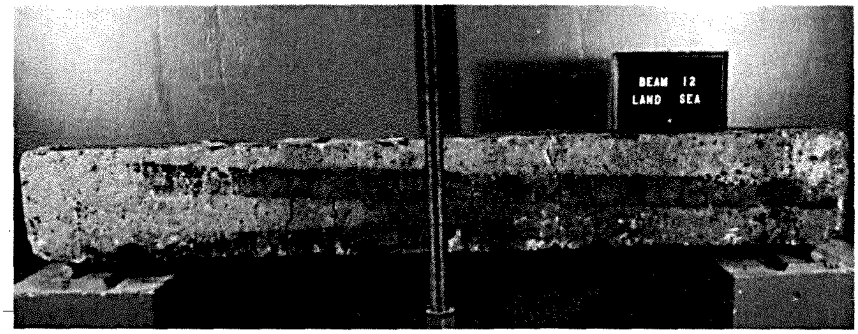


Photo 20.' "As received" condition of beam 12 looking east

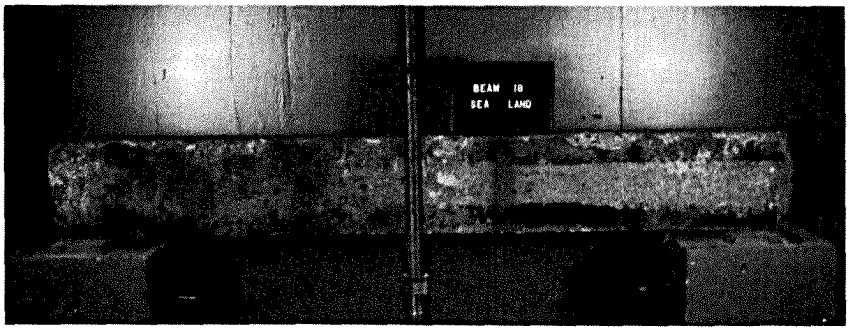


Photo 21. "As received" condition of beam 18 looking west

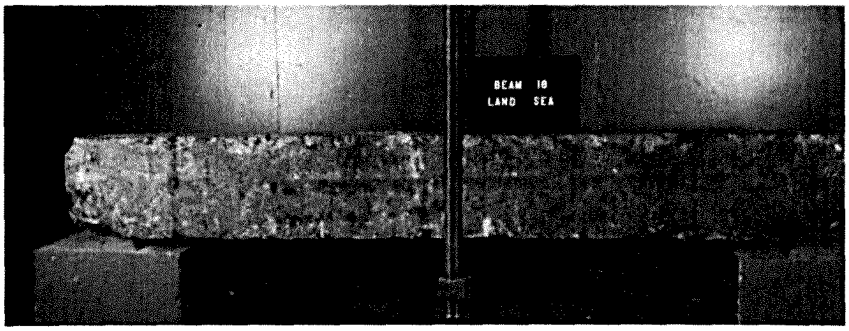


Photo 22. "As received" condition of beam 18 looking east

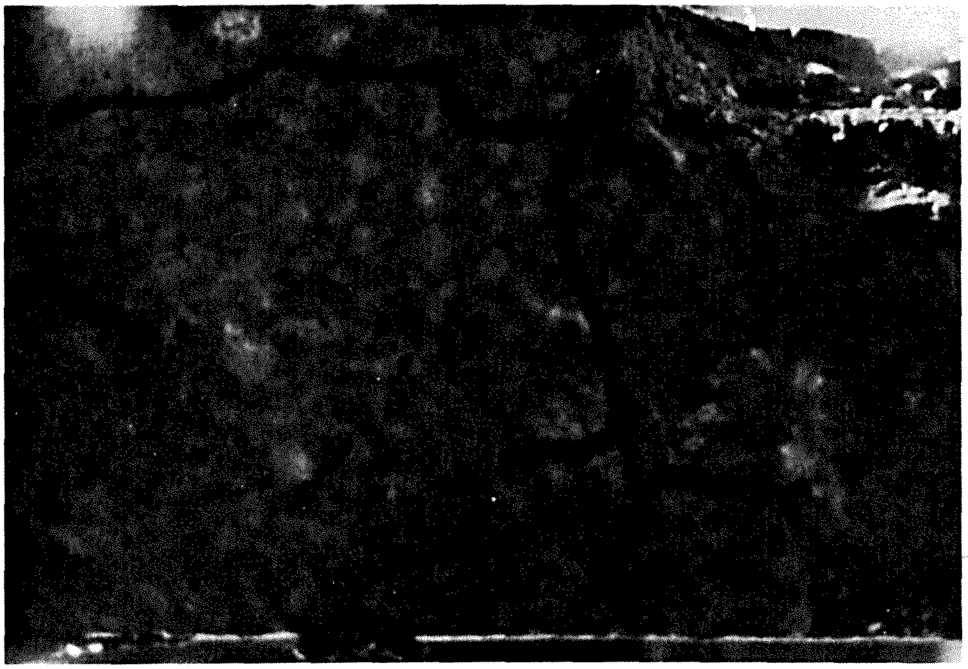


Photo 23. Longitudinal crack on bottom of beam at reinforcement

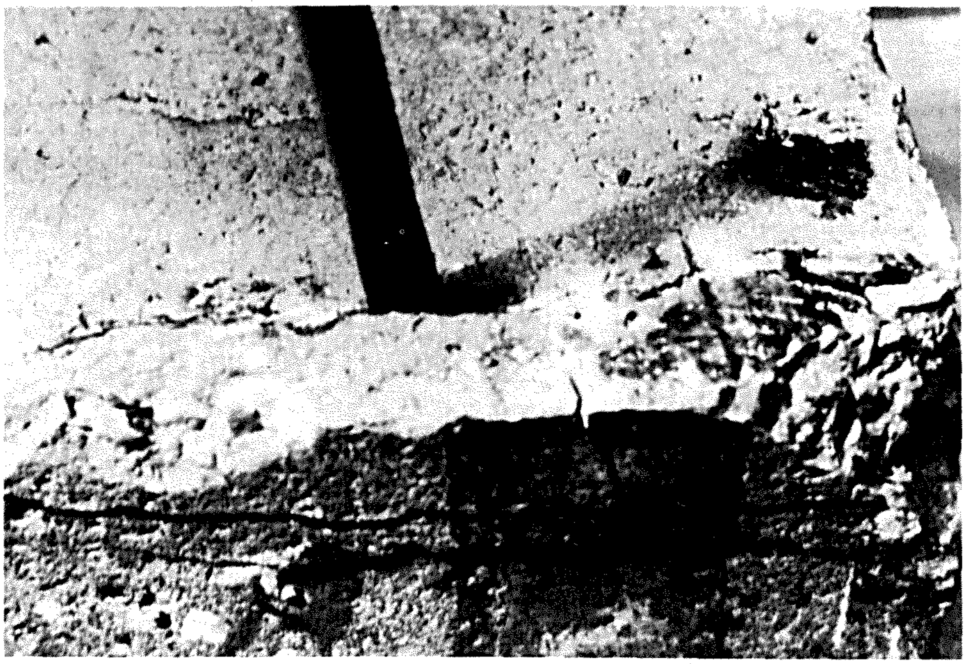


Photo 24. Longitudinal crack on bottom and side of beam at reinforcement

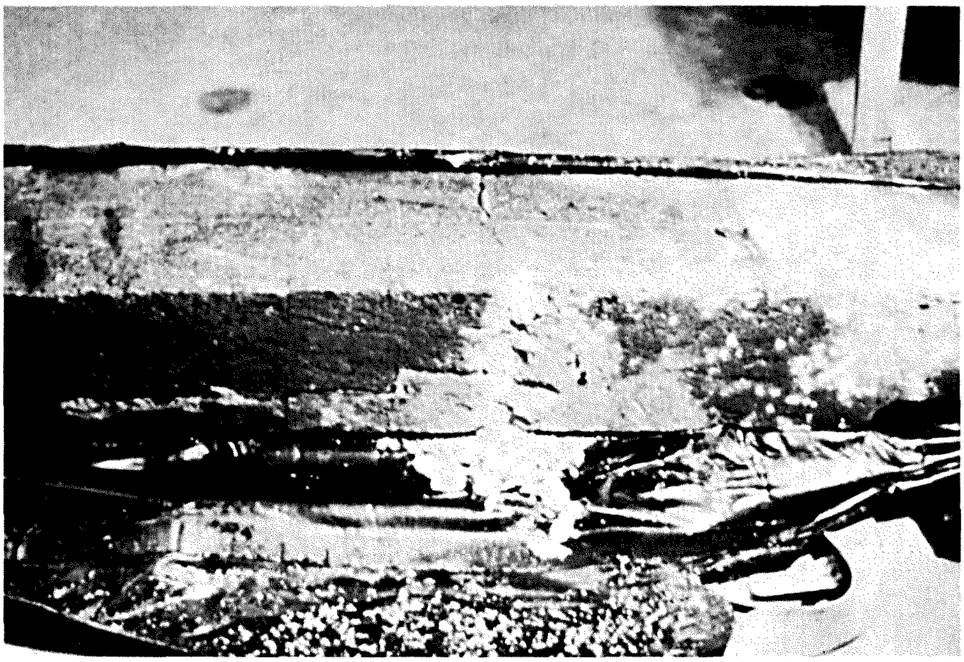


Photo 25. "As received" condition of beams showing failure of the concrete in transit

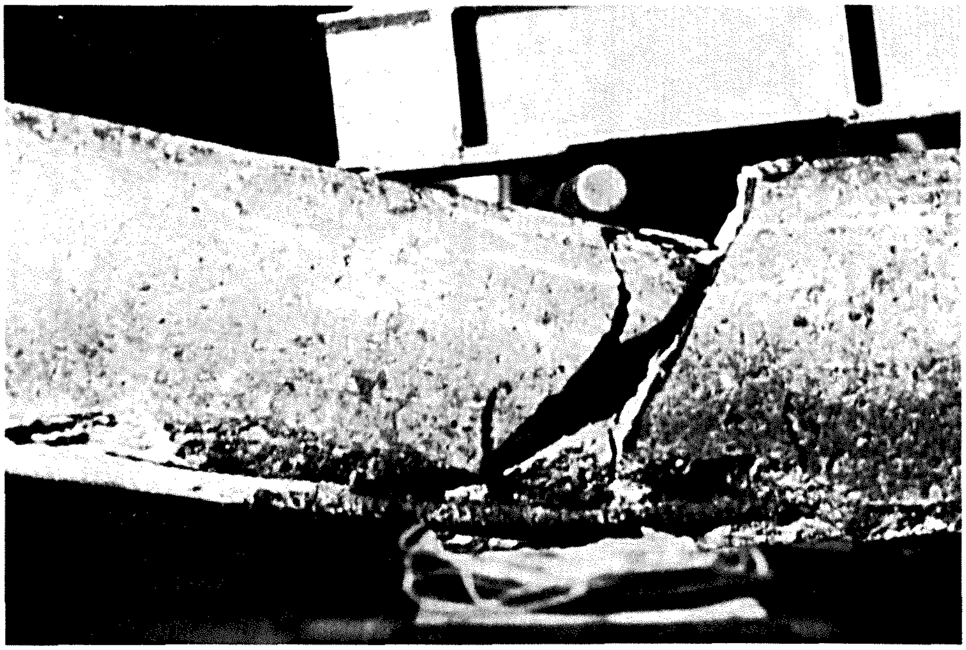


Photo 26. Diagonal tension failure of beam tested in the laboratory

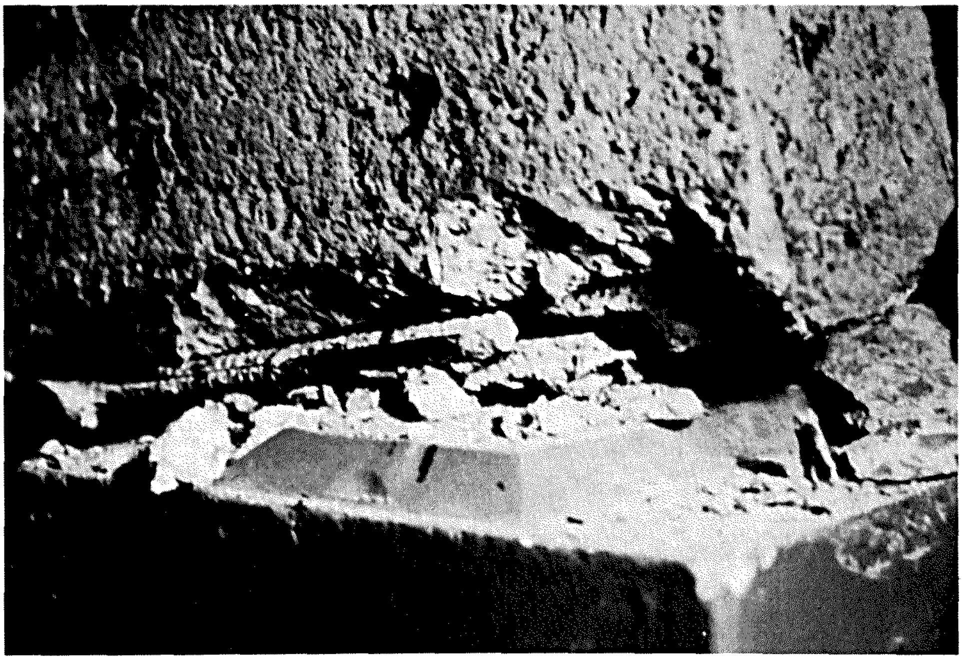


Photo 27. Diagonal tension failure of beam with pullout of reinforcement from the concrete at failure

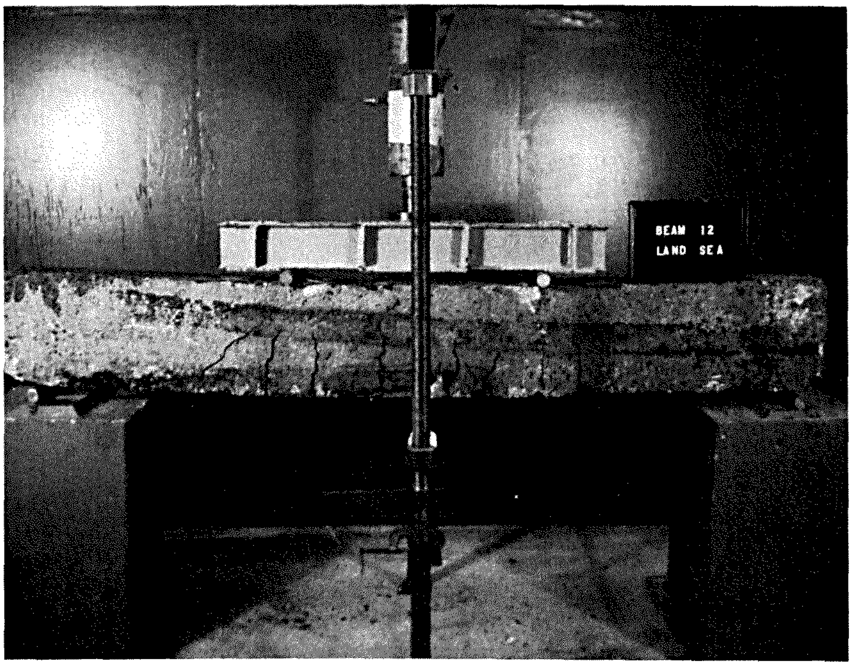


Photo 28. Failure condition of beam not experiencing reinforcement pullout

Photo 29. Imprint of reinforcement bar in cement paste

Photo 30. Corrosion on reinforcement
adjacent to weld

Photo 31. Corrosion on reinforcement at spalled area

Photo 32. Corrosion on reinforcing bar adjacent to spalled area

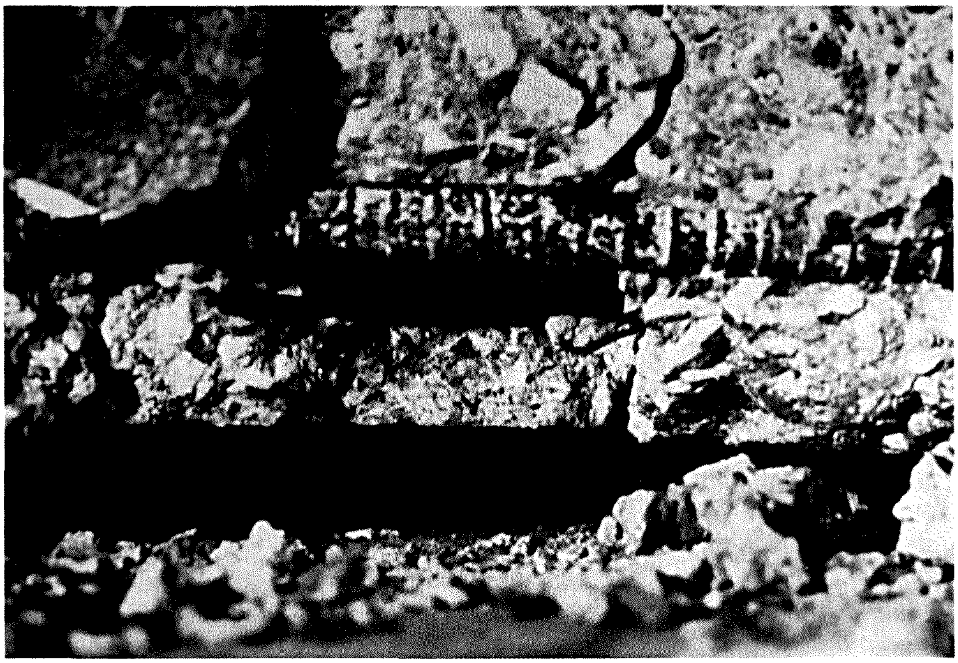


Photo 33. Moderate amounts of corrosion on reinforcement

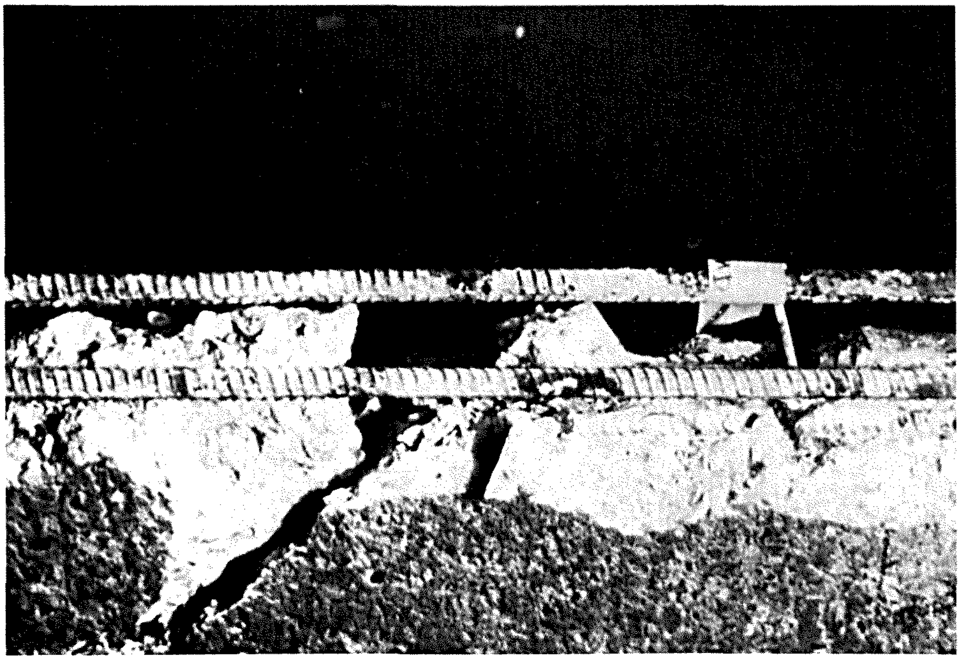


Photo 34. Light corrosion at center of reinforcement bar

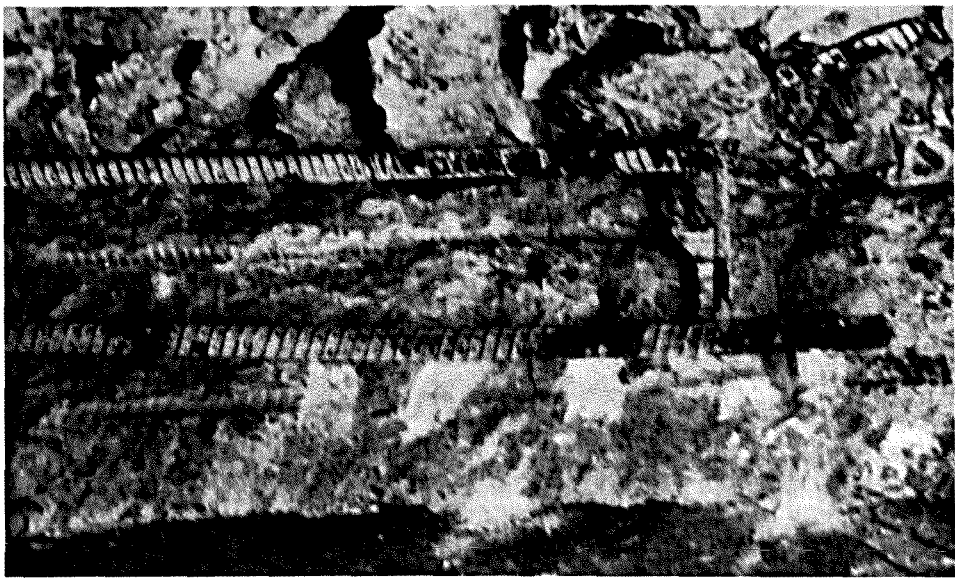


Photo 35. Light corrosion at end of reinforcement bar

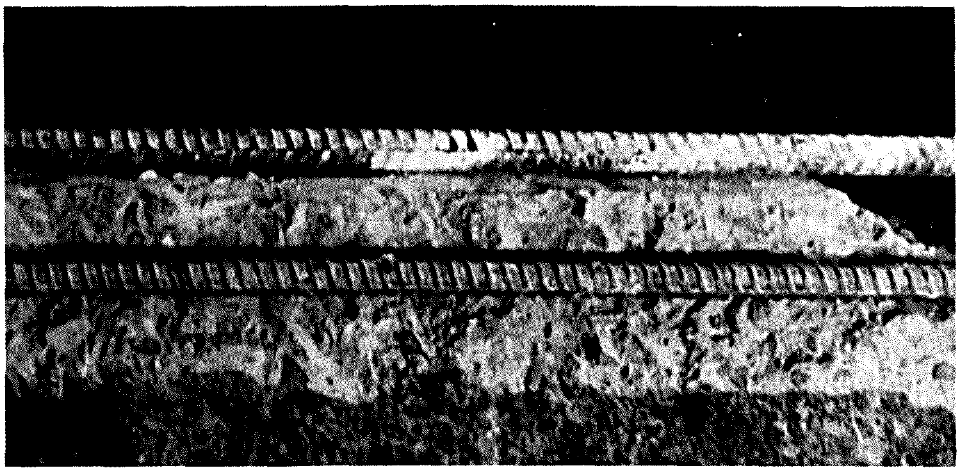
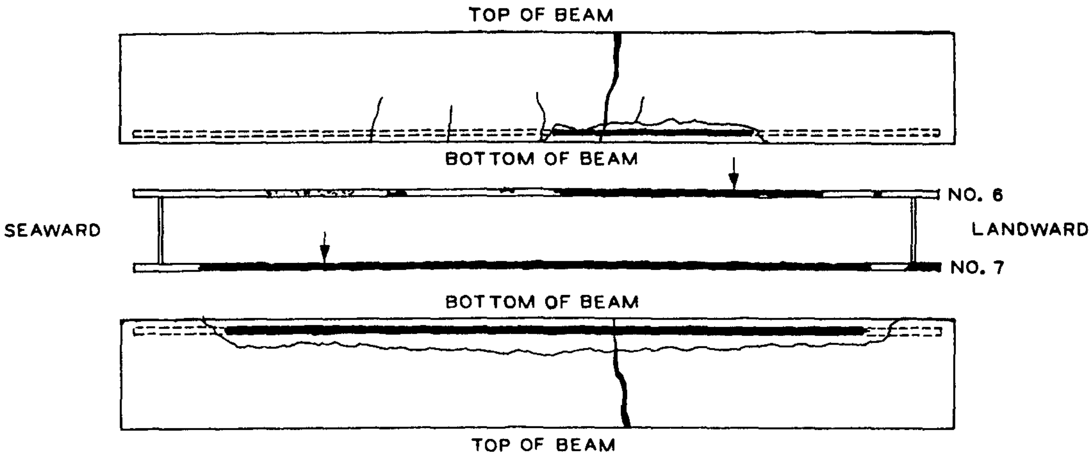


Photo 36. Uncorroded condition of reinforcement bar

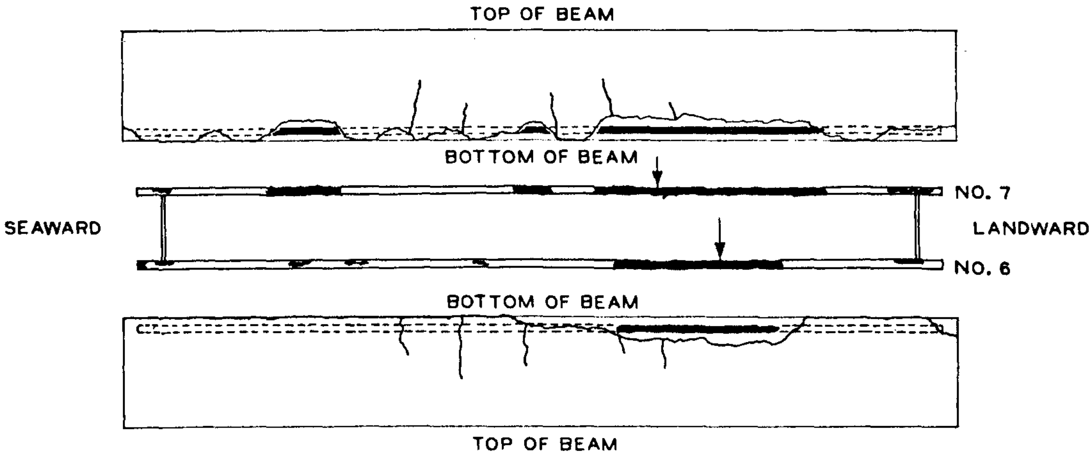
Photo 37. Uncorroded condition of reinforcement bar beneath unspalled concrete

Photo 38. Uncorroded condition of reinforcement bar at a transverse crack



BEAM 1

STEEL STRESS, 20,000 PSI



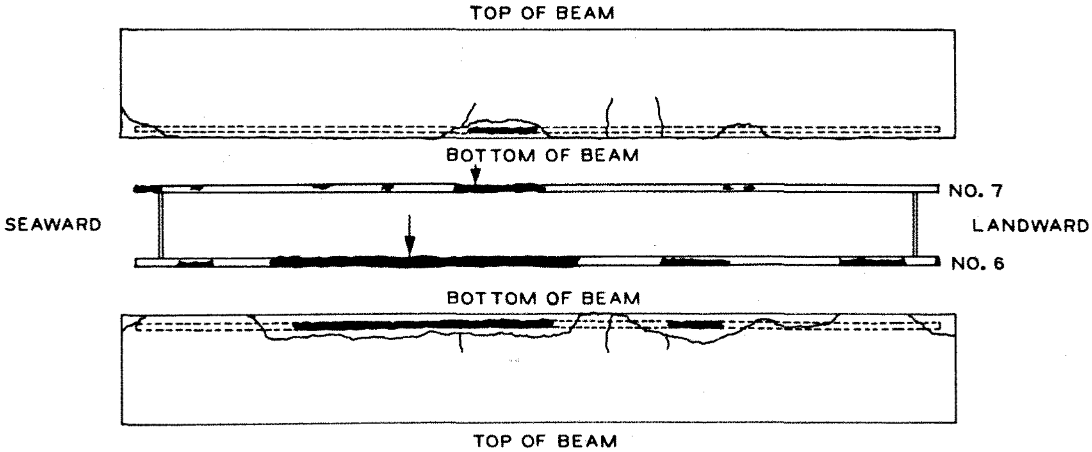
BEAM 2

STEEL STRESS, 20,000 PSI

↓ POINT OF MINIMUM CROSS-SECTIONAL AREA

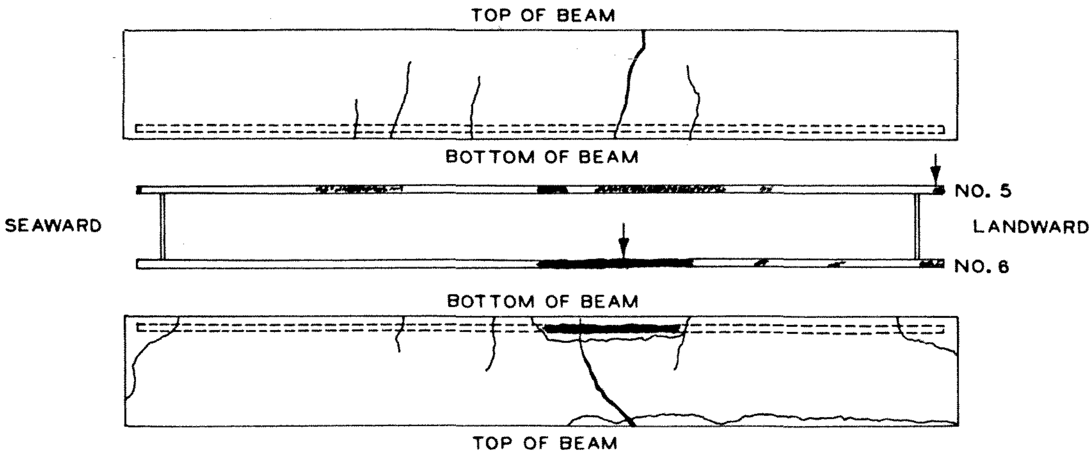
GRAPHIC REPRESENTATION OF CRACKING AND CORROSION

BEAMS 1 AND 2



BEAM 4

STEEL STRESS, 20,000 PSI



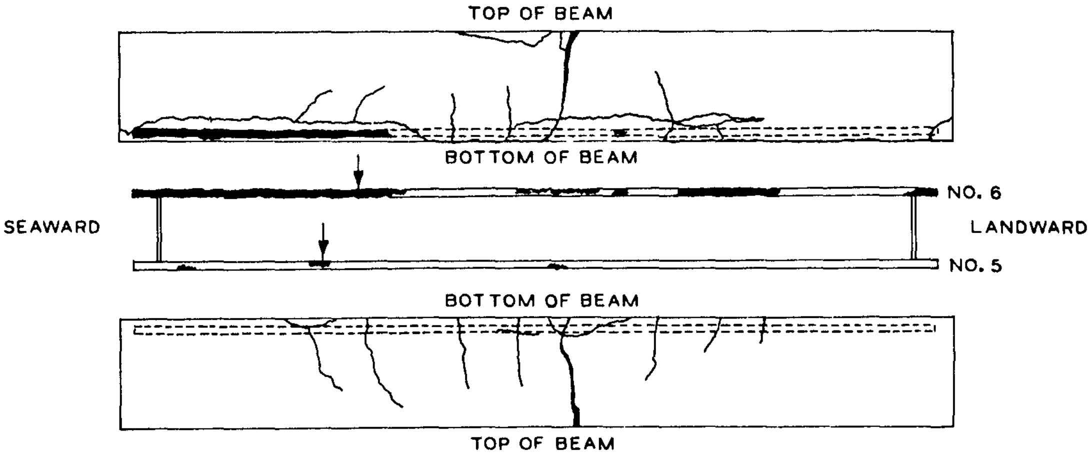
BEAM 5

STEEL STRESS, 30,000 PSI

↓ POINT OF MINIMUM CROSS-SECTIONAL AREA

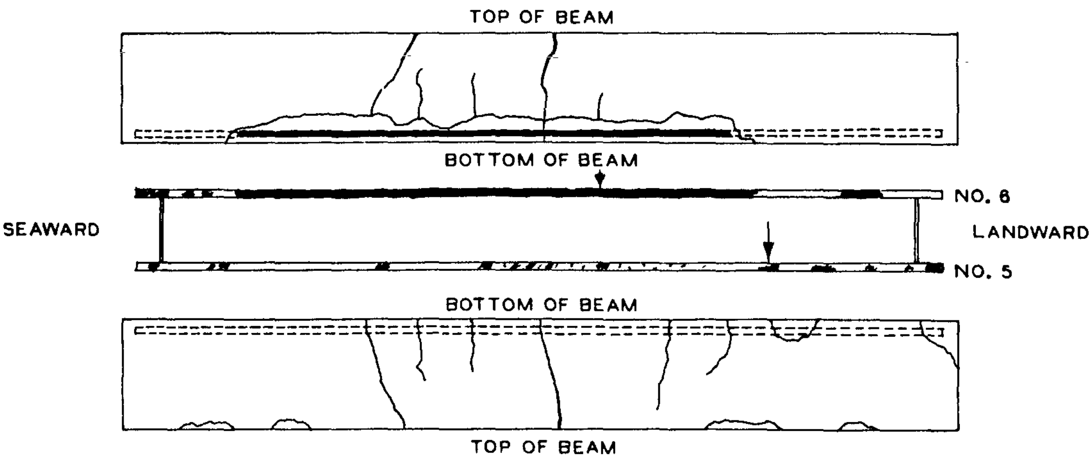
GRAPHIC REPRESENTATION OF CRACKING AND CORROSION

BEAMS 4 AND 5



BEAM 6

STEEL STRESS, 30,000 PSI



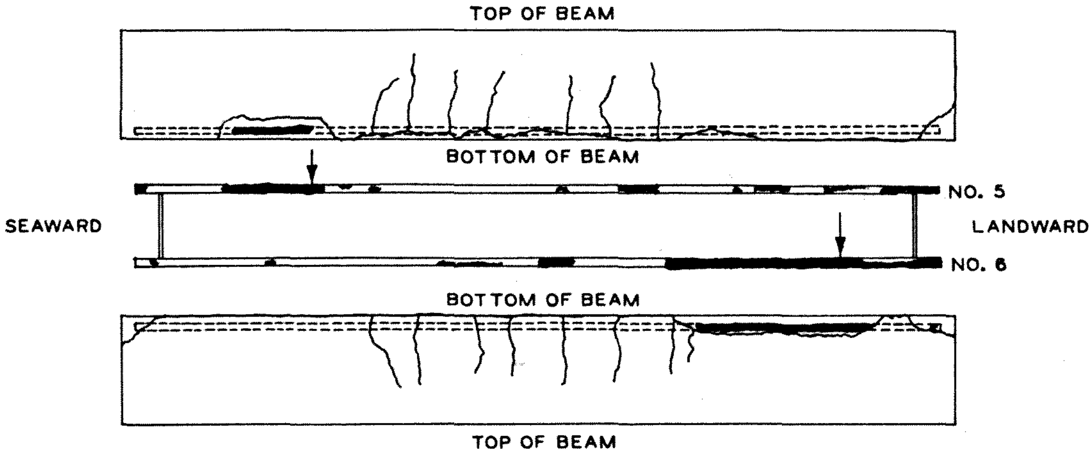
BEAM 7

STEEL STRESS, 30,000 PSI

POINT OF MINIMUM CROSS-SECTIONAL AREA

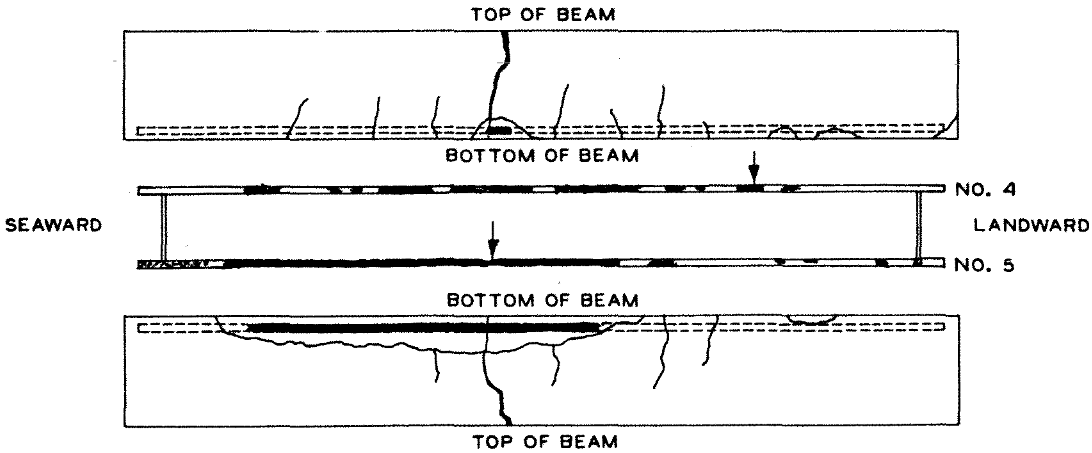
GRAPHIC REPRESENTATION OF CRACKING AND CORROSION

BEAMS 6 AND 7



BEAM 8

STEEL STRESS, 30,000 PSI



BEAM 10

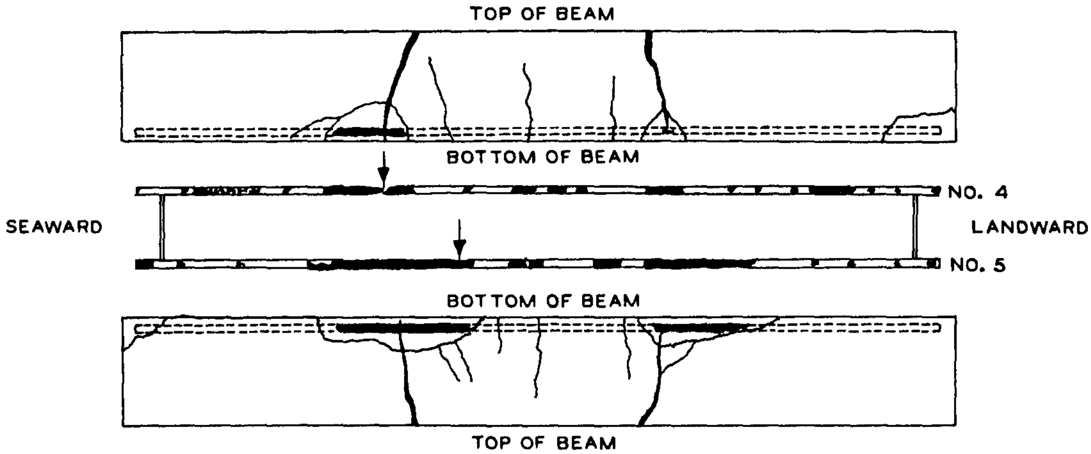
STEEL STRESS, 40,000 PSI

↓ POINT OF MINIMUM CROSS-SECTIONAL AREA

GRAPHIC REPRESENTATION OF CRACKING AND CORROSION

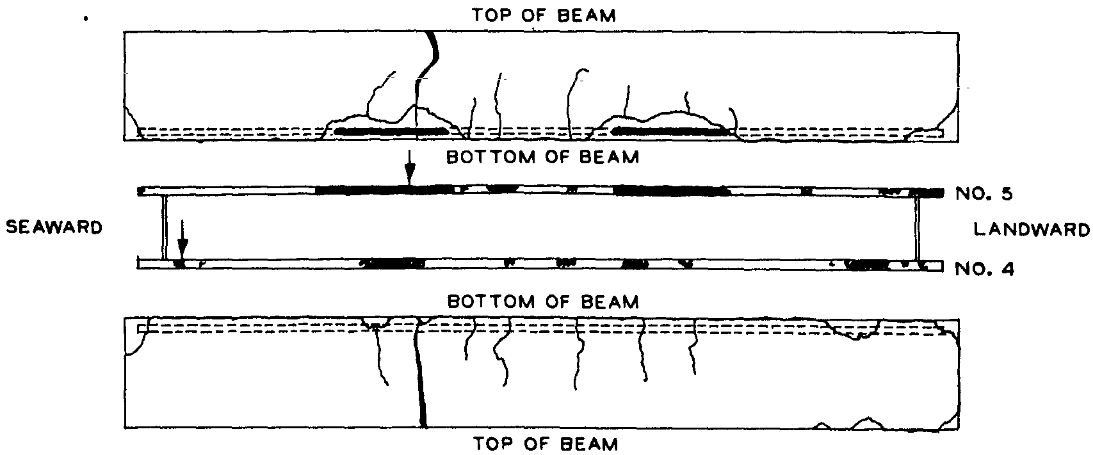
BEAMS 8 AND 10

PLATE 4



BEAM 11

STEEL STRESS, 40,000 PSI



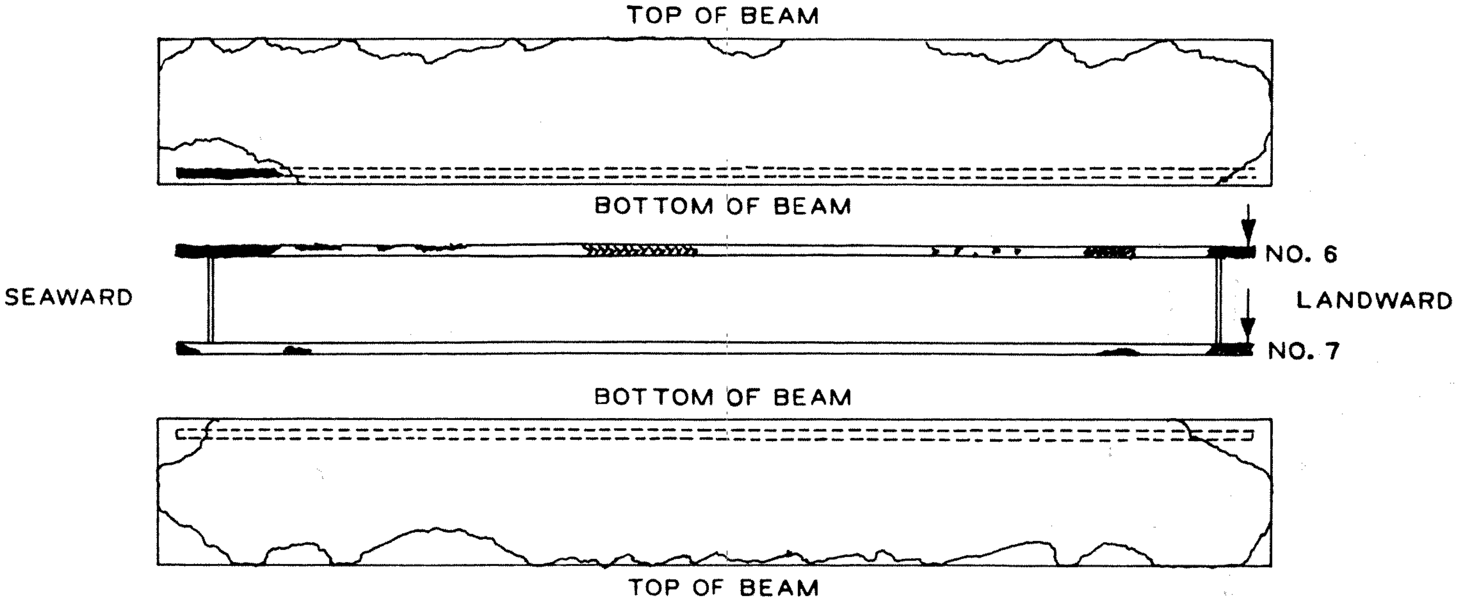
BEAM 12

STEEL STRESS, 40,000 PSI

↓ POINT OF MINIMUM CROSS-SECTIONAL AREA

GRAPHIC REPRESENTATION OF CRACKING AND CORROSION

BEAMS 11 AND 12



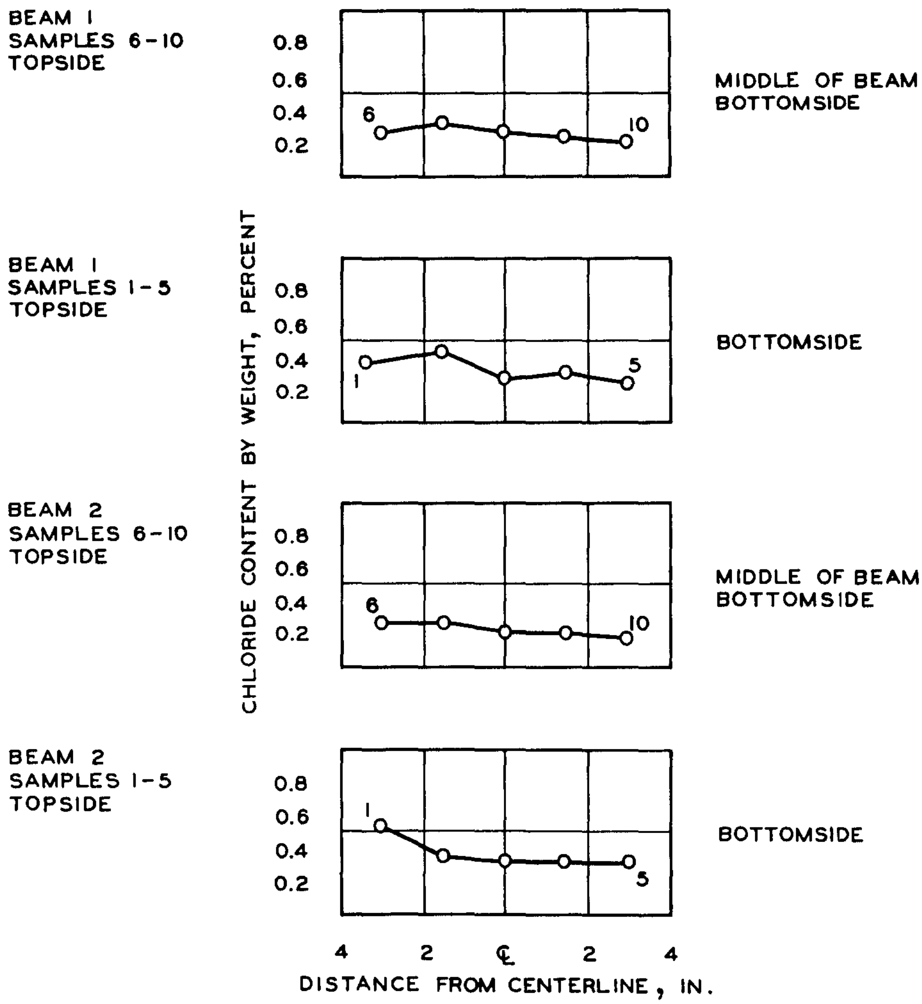
BEAM 18

CONTROL, STEEL UNSTRESSED

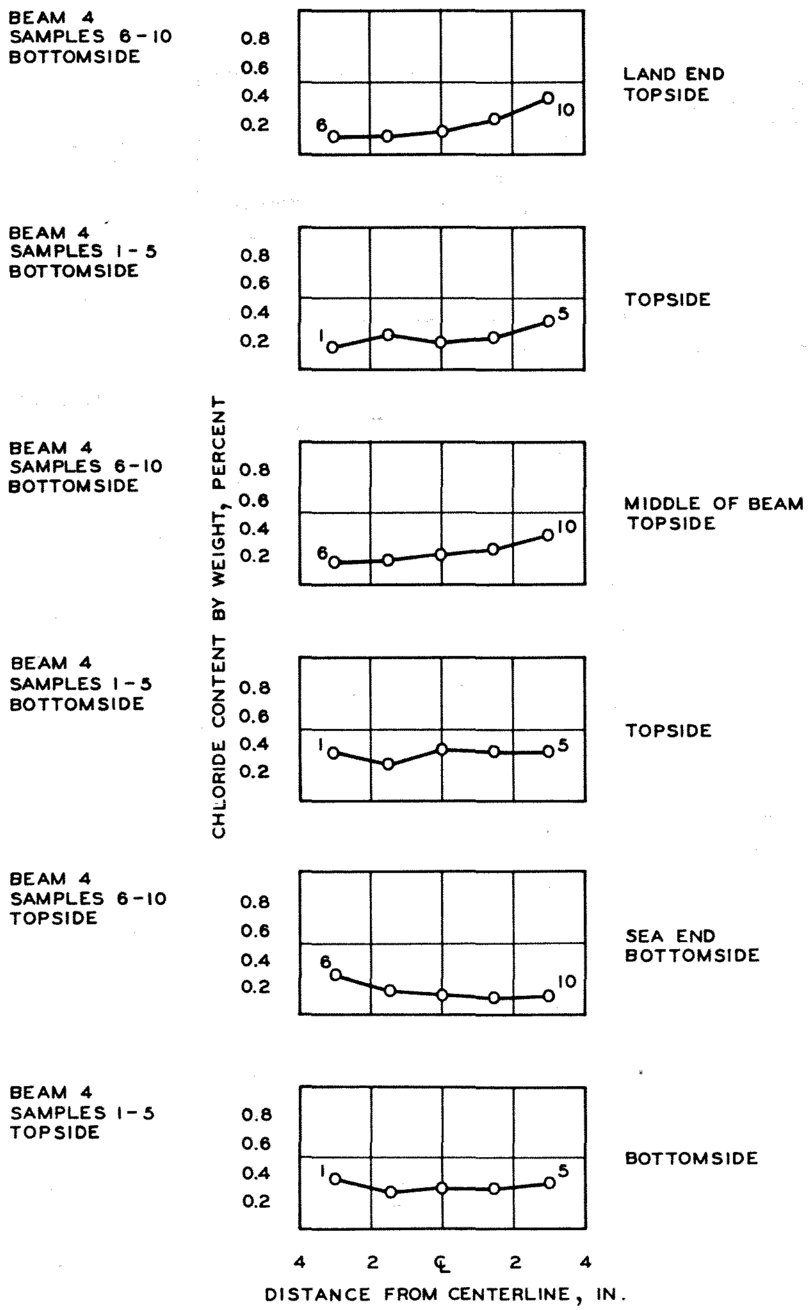
↓ POINT OF MINIMUM CROSS-SECTIONAL AREA

GRAPHIC REPRESENTATION OF CRACKING AND CORROSION

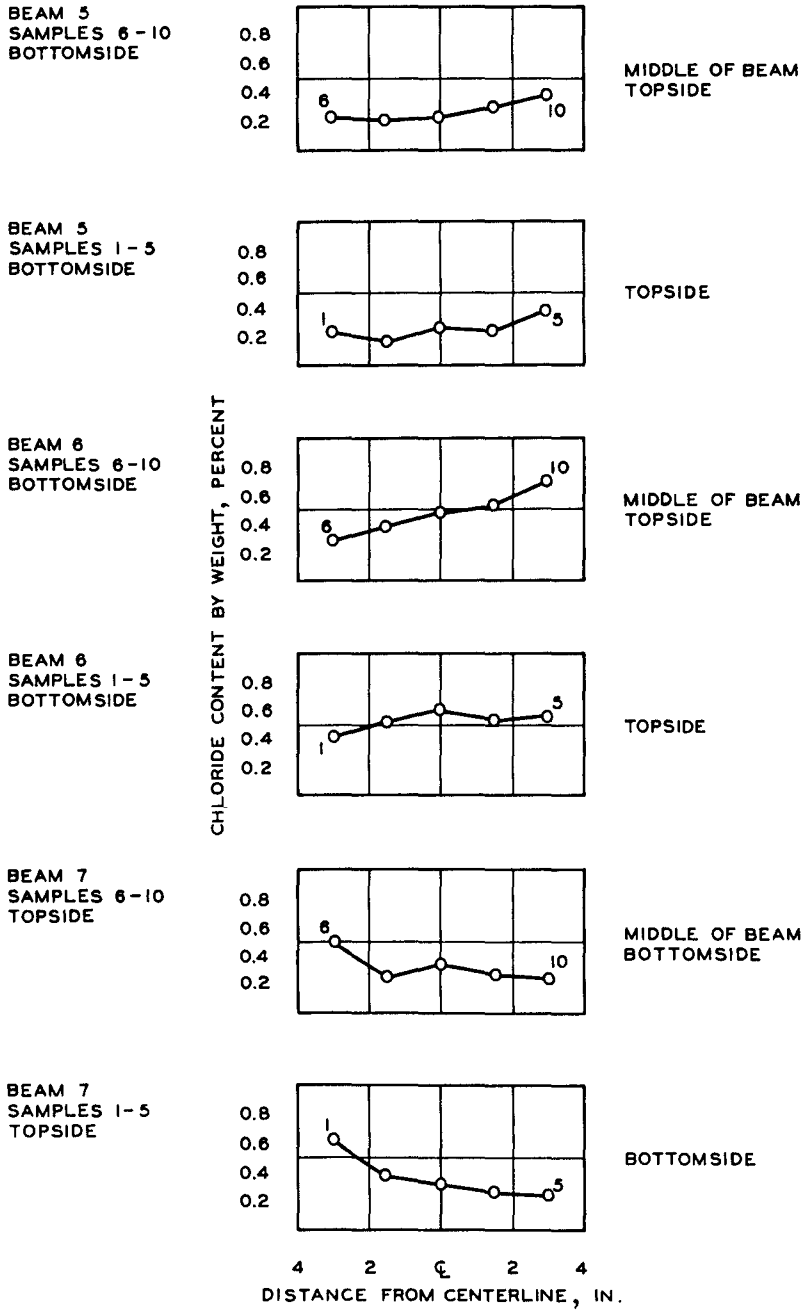
BEAM 18



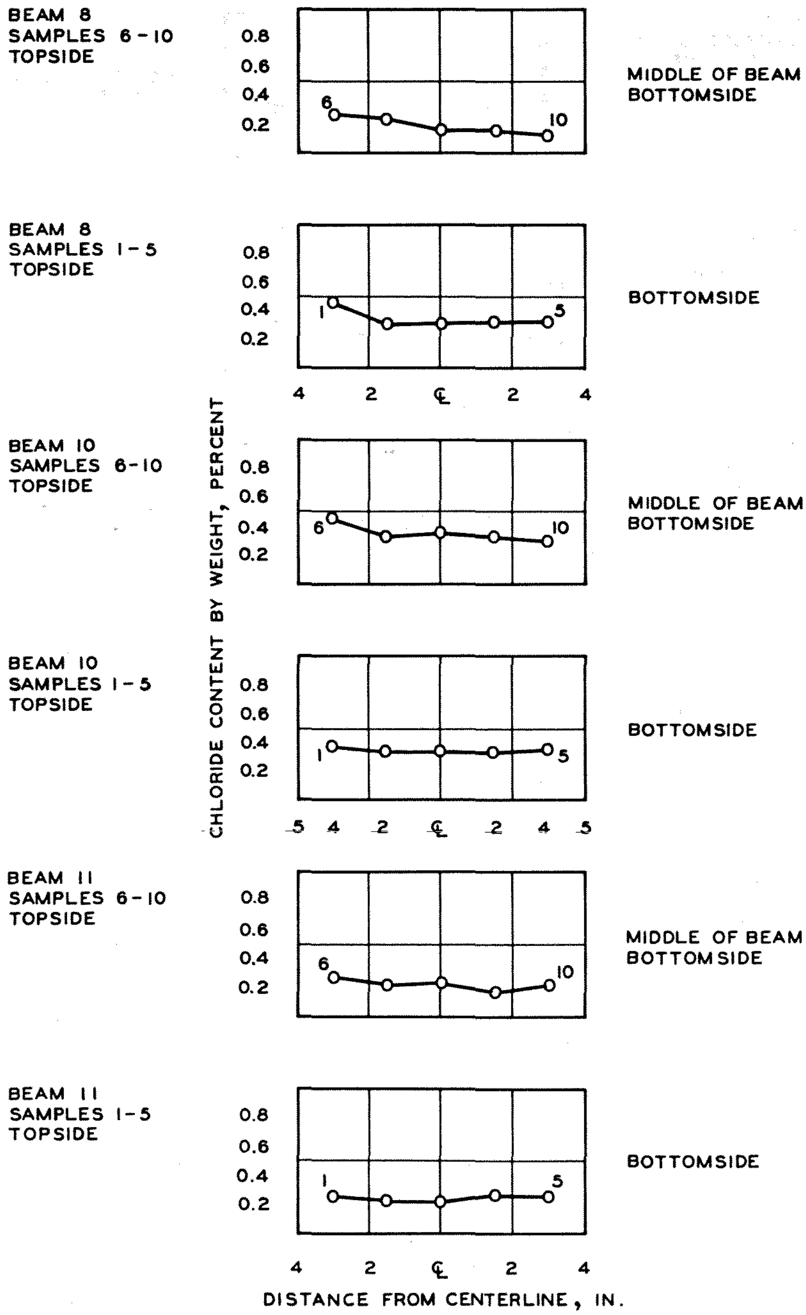
CHLORIDE CONTENT, %
VS DEPTH FROM SURFACE
OF BEAMS 1 AND 2



CHLORIDE CONTENT, %
VS DEPTH FROM SURFACE
OF BEAM 4

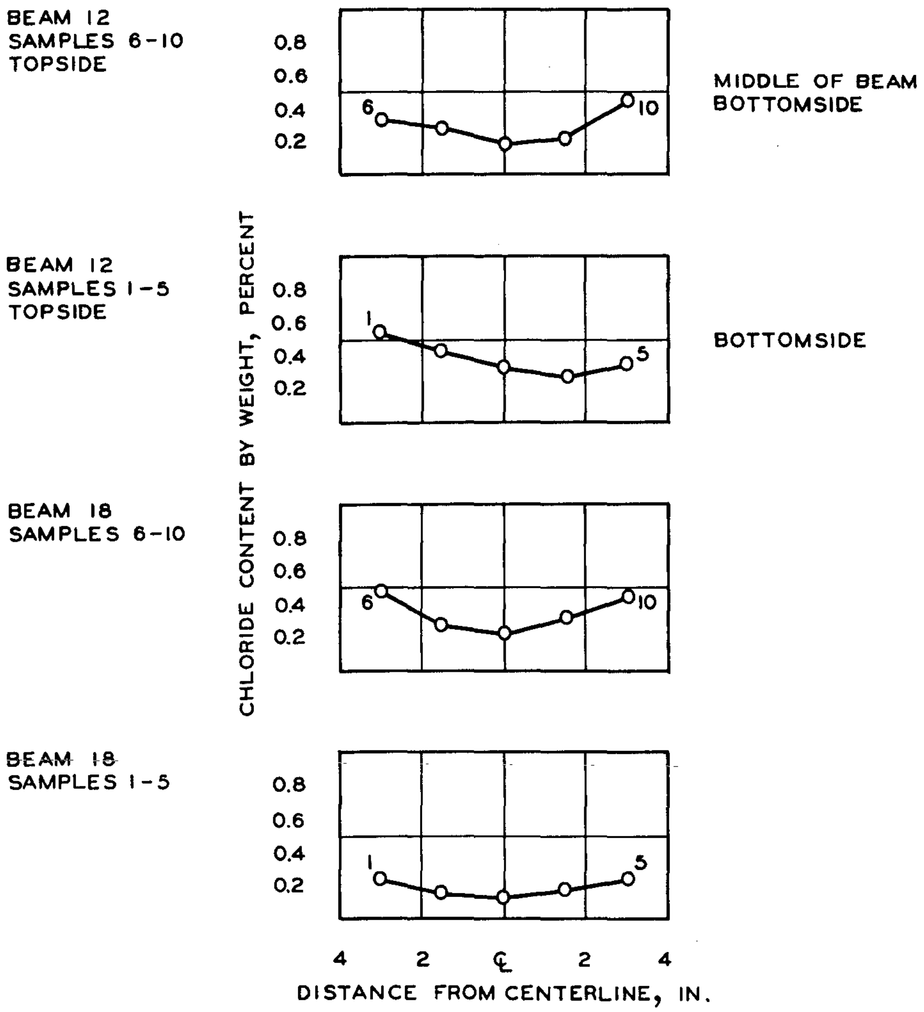


CHLORIDE CONTENT, %
VS DEPTH FROM SURFACE
OF BEAMS 5, 6 AND 7



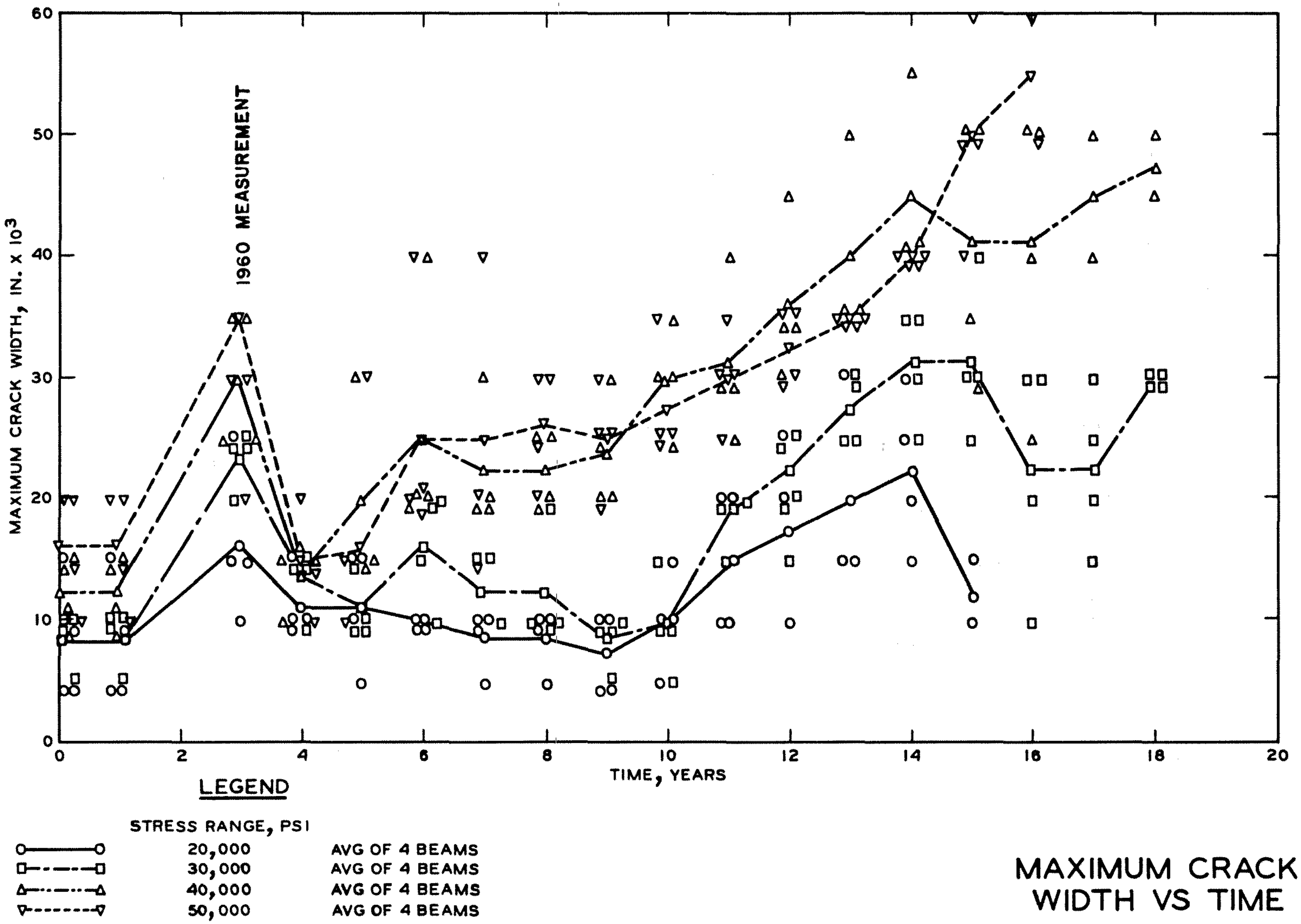
CHLORIDE CONTENT, %
VS DEPTH FROM SURFACE
OF BEAMS 8, 10 AND 11

PLATE 10

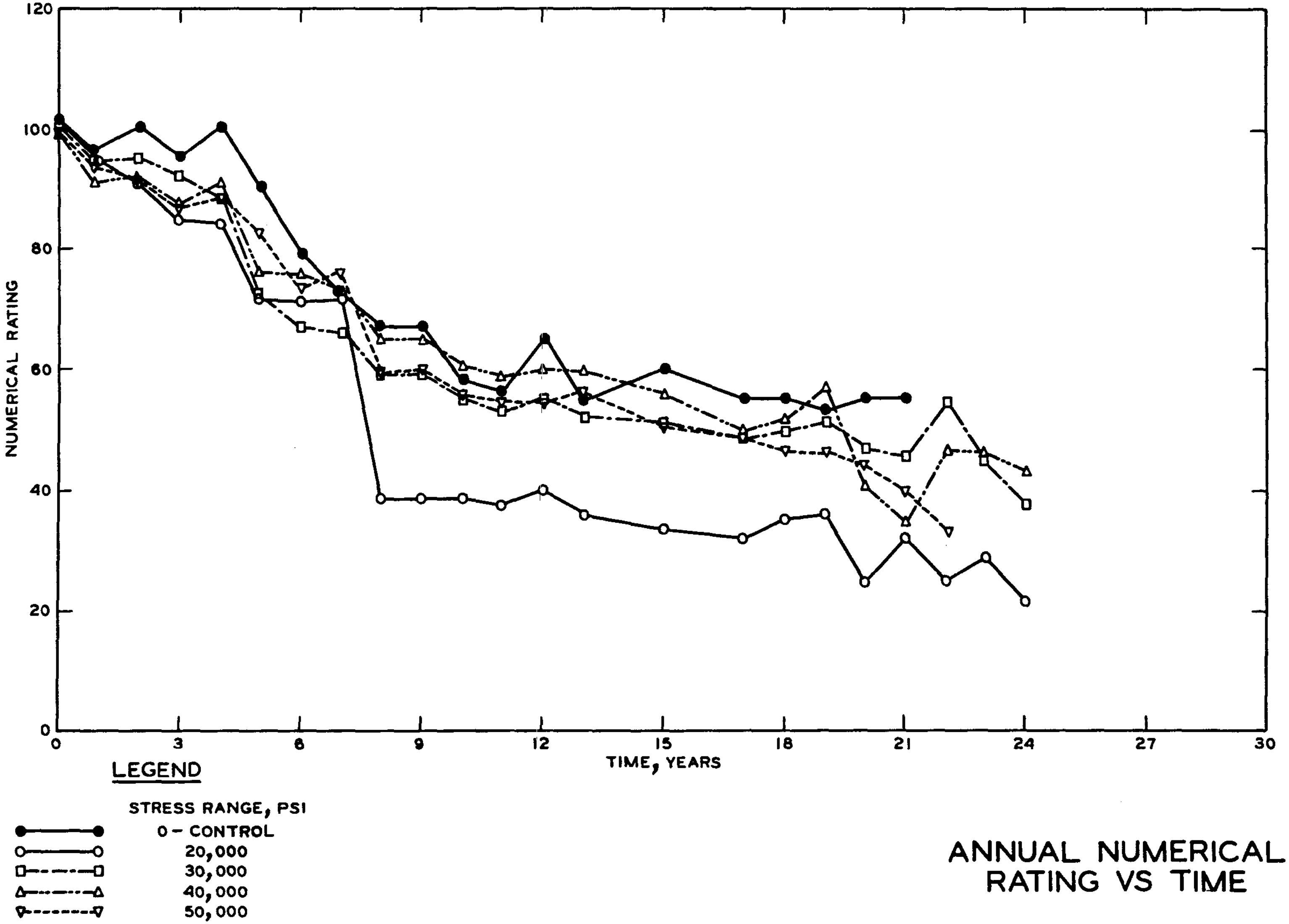


CHLORIDE CONTENT, %
VS DEPTH FROM SURFACE
OF BEAMS 12 AND 18

PLATE 12



MAXIMUM CRACK WIDTH VS TIME



ANNUAL NUMERICAL RATING VS TIME

PLATE 13

APPENDIX A: SAMPLE INSPECTION SHEET

Inspection Sheets
Formal Inspection, Treat Island, Maine

Tensile Crack Exposure Tests Date_____________

Instructions:

1. Insert in column headed, "No. of transverse cracks with spalling," the number of load cracks that have apparently chipped or spalled subsequent to formation when beams were loaded, that now have places in which a pencil can be inserted (about 1/4 in. wide).

2. Measure (Note) the total length of cracking in inches appearing over the reinforcing steel.

3. Measure the total length of reinforcement that can be seen through cracks, or that is exposed because concrete has spalled away from it.

4. Measure the total length of cracking bordered by iron stain from the crack.

5. Estimate the total area of visible horizontal and vertical surface of concrete that has scaled and make a check under the most appropriate heading on the rating sheet.

Note: Measure to $\pm$ 1/4 in.

Scoring:

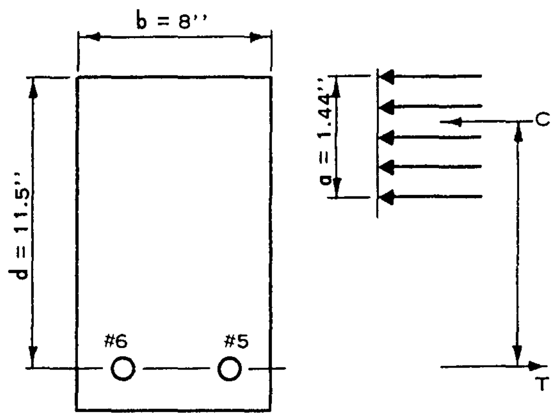
a. Scoring will be done using a numerical system by others after the inspection.

b. Score of zero indicates perfect condition.

c. Light scaling scores 2, medium scaling 4, heavy scaling 8.

d. Numerical score = sum of 4 x number of spalled cracks + length of cracking over steel + 3 x length of visible steel + length of cracking over steel bordering iron-stained areas + appropriate score for scaled area.

APPENDIX B: SAMPLE ULTIMATE DESIGN MOMENT CALCULATION



$A_s = 0.31 + 0.44 = 0.75 \text{ in.}^2$
$f'_c = 3.820$ ksi
$f_y = 50.00$ ksi
$k_1 = 0.85$ ($f'_c = 3.820$)
Section is underreinforced.

For equilibrium just before ultimate load

$$C = T = A_s f_y = (0.75)(50.0) = 37.5 \text{ kips}$$

solving for the depth of the compression block

$$a = \frac{C}{0.85 f'_c b}$$

$$= \frac{37.5}{(0.85)(3.82)(8)}$$

$$= 1.44 \text{ in.}$$

and

$$\frac{a}{2} = 0.72 \text{ in.}$$

then the ultimate moment is

$$M'_u = C\left(d - \frac{a}{2}\right)$$

$$= \frac{37.5(11.5 - 0.72)}{12}$$

$$= 33.68 \text{ ft-kips}$$

The ratio of the actual ultimate moment to design ultimate moment is, for beam 5:

$$P_{ult} = 34.7 \text{ kips}$$

$$M_u = \frac{P_{ult}}{2}\left(\frac{\ell}{3}\right) = \frac{34.7}{2}(2.36) = 40.95 \text{ ft-kips}$$

$$\text{Ratio} = \frac{40.95}{33.68} = 1.216$$

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

O'Neil, Edward F

Tensile crack exposure tests; Report 3: Laboratory evaluation of Series "A" beams with results from 1951 to 1975 / by Edward F. O'Neil. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1980.

31, [24] p., 7 leaves of plates : ill. ; 27 cm. (Technical memorandum - U. S. Army Engineer Waterways Experiment Station ; 6-412, Report 3)

Prepared for Office, Chief of Engineers, U. S. Army, Washington, D. C., under Civil Works Research Work Unit 010401/31276.

References: p. 31.

1. Air entrained concretes. 2. Concrete beams. 3. Concrete cracking. 4. Concrete exposure. 5. Freeze-thaw durability. 6. Reinforced concrete. 7. Tensile strength (Concrete). 8. Weathering (Concrete). I. United States. Army. Corps of Engineers. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical memorandum ; 6-412, Report 3.

TA7.W34 no.6-412 Report 3