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PROTECTIVE SYSTEMS FOR NEW PRESTRESSED AND SUBSTRUCTURE CONCRETE

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FOREWORD

This report will be of interest to bridge engineers and designers of conventionally reinforced and prestressed concrete structures exposed to salts from winter deicing or from ocean marine environments.

Eleven corrosion protection systems utilizing different concrete water/cement ratios, different clear covers over the embedded steels, epoxy-coated bars and prestressing strands, galvanized bars, conventional bars and prestressing strands, a calcium nitrite corrosion-inhibiting admixture, a silane surface sealer, a methacrylate coating for concrete, and a concrete containing a silica fume admixture were evaluated. The tests were performed in two separate one-year long accelerated laboratory investigations which utilized realistic saltwater exposure conditions. These tests initially evaluated 124 small slab specimens followed by 19 full-size sections of columns, beams, prestressed piles and stay-in-place precast, prestressed bridge deck panels. These were evaluated using selected corrosion protection systems and materials which had shown the most promise in the small slab test program.

Richard E. Hay
Director, Office of Engineering
and Highway Operations
Research and Development

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16. Abstract

A three-year corrosion research project on 11 corrosion protection systems was undertaken in two laboratory studies. A total of 124 small reinforced concrete slabs were subjected to a 48-week, cyclic wet and dry saltwater exposure in the first study. The second year-long study dealt with cyclic saltwater exposure on 19 full-size sections of reinforced concrete columns and beams and precast, prestressed piles and stay-in-place bridge deck panels.

The slab tests evaluated concrete having w/c ratios of 0.51, 0.40 and 0.28 with clear cover of 1, 2 and 3 in. The reinforcing steels evaluated were normal gray bars, normal prestressing strands, galvanized bars, and fusion-bonded epoxy-coated bars and prestressing strands. A calcium nitrite corrosion-inhibiting admixture for fresh concrete was evaluated. A penetrating silane sealer and a methacrylate coating system were evaluated as surface treatments for hardened concrete. The full-size member tests included most of the above materials in concrete at a constant w/c ratio of 0.44, generally with 1-in. cover. A concrete containing a silica fume admixture was also evaluated in the full-size member tests. The full-size columns and beams were moist cured while the precast, prestressed piles and bridge deck panels were heat cured overnight at 130 to 140°F.

Corrosion-related measurements included monitoring macrocell corrosion current and instant-off voltage between corroding and noncorroding reinforcement, half-cell potentials, chloride contents at the initiation of corrosion and at the conclusion of the test cycle, and measurement of the corroded areas on the reinforcement.

The measured corrosion activity of these numerous specimens is presented and comparisons are made in their corrosion protection performance. Of particular significance were the beneficial influence of low w/c ratios and adequate concrete cover in reducing corrosion and chloride penetration. The silane sealer and the silica fume pozzolanic admixture concrete both dramatically reduced chloride penetration and the reinforcement did not develop corrosion. Epoxy-coated reinforcing bars and prestressing strands were not corroded in these tests, even though surrounding concrete had high chloride contents. Galvanized reinforcement embedded in conventional concrete and bare reinforcement embedded in concrete made with the calcium nitrite corrosion inhibitor both developed a low level of corrosion when surrounding concrete had high chloride contents. Design considerations are suggested for these different corrosion protection materials to provide for greater corrosion protection for new cast-in-place reinforced concrete and precast, prestressed concrete bridge members.

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METRIC CONVERSION FACTORS

Conversion Factors - U.S. Customary to SI (Metric)

1 in = 25.4 mm
1 ft = 0.305 m
1 yd = 0.914 m

1 fluid oz = 29.57 cm^3
1 gallon = 0.003785 m^3
1 in^3 = 16.4 cm^3
1 ft^3 = 0.0283 m^3
1 yd^3 = 0.765 m^3

1 lb (force) = 4.448 N
1 lb (mass) = 453.6 g

1000 psi = 6.895 M Pa

1 lb/ft^3 = 16.02 Kg/m^3
1 lb/yd^3 = 0.5933 Kg/m^3
1 bag/yd^3 = 94 lbs/yd^3 = 56.4 Kg/m^3

Degree C = 5/9 (Degree F-32)

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INTRODUCTION

For many years, the corrosion resistance of reinforced concrete highway structures was assumed to be adequate because of the demonstrated corrosion-inhibiting properties of portland cement concrete. Because of this and an underestimation of the impact on corrosion caused by the gradual ingress of chloride ion from deicing salts, the AASHTO code requirements were unchanged for many years. A review of the AASHTO codes from 1931 to 1983 shows the following significant observations related to reinforcement corrosion protection through concrete quality and clear cover requirements.

- Reinforced concrete exposed to seawater has required a 4-in. clear cover for over 40 years.
- Precast concrete piles exposed to seawater have required a 3-in. clear cover for over 40 years.
- Reinforced concrete exposed to normal weathering generally required a 2-in. clear cover and a 3000 psi 28-day compressive strength until 1977. Then the required compressive strength was increased to 4000 psi for non-air-entrained concrete and 4500 psi for air-entrained concrete.
- Slabs for bridge decks required 1-in. minimum clear cover until 1969, when the top-of-slab bars required 1 1/2-in. cover. In 1974, code requirements for top-of-slab bars increased to 2 in. Required bottom-of-slab bar cover has remained at 1 in. for 50 years.
- From 1953 to 1973, concrete w/c ratio was not specified for Class A or Class A (AE) concretes. Instead, a minimum cement content of 6 bags was required with a 28-day strength of 3000 psi. In 1977, the minimum cement content was increased to 6.5 bags for Class A or A (AE) concretes, and the specified w/c ratios and compressive strengths for Class A and A (AE) concretes were 0.49 and 0.44, and 4000 psi and 4500 psi, respectively. This was the first major revision of AASHTO requirements in many years that stressed lower permeability concrete.

AASHTO currently requires high quality, more impervious concrete than previously, with specific recommendations for more clear cover when it is in a corrosive environment. The current w/c ratio requirements for air-entrained concrete exposed to chloride ions is 0.44. Clear cover requirement ranges from 1.0 to 4.0 in. depending upon bar size, environment, steel type, and concrete quality differences between precast and cast-in-place construction techniques.

The corrosion problems of bridge substructures are severe and are similar to those associated with bridge decks. Many of the same factors which lead to corrosion in bridge decks(1-22) also cause corrosion in other concrete bridge members. While most substructures are built using conventionally reinforced concrete, some substructure elements and bridge deck elements are made with precast, pretensioned, prestressed members which may have similar corrosion problems.

Corrosion Protection Systems and Economics

Structural engineers conforming to the AASHTO recommendations for additional clear cover will produce structural designs, utilizing conventional reinforcing steels, which are somewhat heavier but with more corrosion protection than in the past. Numerous new corrosion protection materials(9,14-22) have been developed in the past 10 to 20 years to assist in maintaining structural design efficiency (i.e., to maintain a minimum clear cover) while providing increased corrosion protection. These materials are relatively expensive when compared with the basic costs of portland cement concrete and gray reinforcing steel. Current typical cost premiums for some of these new corrosion protection materials are as follows:

- Fusion bonded epoxy-coated reinforcing bars cost from 30 to 110 percent more than gray bars, depending upon bar size, length, and bending requirements.
- Fusion bonded, epoxy-coated, 7-wire prestressing strands cost about 100 percent more than bare strands.
- Low permeability concretes consisting of low slump, low w/c ratio (0.32) concrete or higher slump, low w/c ratio (0.32)

concrete with Type F high-range water reducers cost from 15 to 30 percent more than a 6-bag mixture.

- Surface-penetrating sealers and coatings can be used to minimize chloride ion ingress into concrete. Particularly good sealers can decrease chloride ingress into concrete by 75 to 95 percent. Chemical formulations(17) with excellent laboratory performance can be applied for some \$0.35 to \$0.75/ft^2 including preparation costs, materials, and labor.
- Corrosion-inhibiting admixtures(15,19) such as calcium nitrite, are being added to the fresh concrete to provide corrosion protection at a cost premium of about 50 percent of the base delivered concrete.

It is reasonable to assume that these materials, which were developed to deal with bridge deck deterioration, will be readily applicable to substructure and precast, pretensioned concrete members, which are the subject of this research.

Development of the Current Test Procedure

Previous laboratory studies(16,19) have shown that a prime cause of corrosion is macroscopic galvanic interaction between corroding "anodic" bars in concrete heavily contaminated with chloride, combined with "cathodic" bars in concrete that contains little chloride. In the laboratory it was possible to create similar macro corrosion cells on a smaller scale and study corrosion parameters under controlled conditions.

Earlier research has identified a "chloride ion corrosion threshold" value of about 1.0 to 1.6 lbs of acid-soluble chloride ion per cu yd of concrete.(8,11,12) With this threshold number identified, it was possible to concentrate on testing corrosion protection materials or systems.

Before undertaking corrosion tests on full-size substructure and prestressed bridge members to determine optimum parameters for corrosion protection in high chloride environments, it was decided to make comparisons between available corrosion protection systems during Pilot

Time-to-Corrosion accelerated laboratory tests. A schematic of the concrete specimen chosen for these pilot tests is shown in Fig. 1.

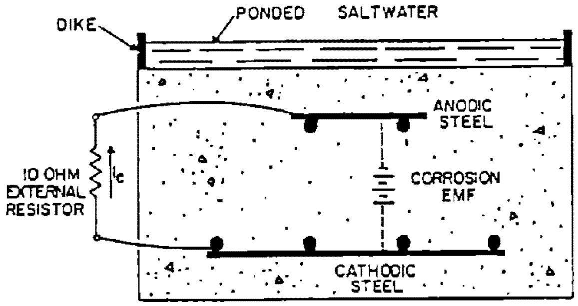


Fig. 1 - Fabricated standard concrete specimen for corrosion monitoring.

The specimen contains an upper mat of "anodic steel" in a concrete that gradually absorbs moisture and chloride from cyclic ponding of saltwater. The lower steel remains in a relatively chloride-free concrete environment and becomes cathodic. The ends of the upper and lower mats of steel bars are connected externally so that the flow of corrosion current, I_c, between the cathodic and anodic levels can be measured. Other corrosion parameters, such as instant-off potentials, (I.O.P.), and concrete electrical resistance can also be measured. These types of measurement have been reported earlier.(16,19) Half-cell potentials at the top surfaces were measured by a standard ASTM procedure.(23)

Possible uses of this test procedure and specimen configuration are:

1. The effects of variables such as clear cover, w/c ratio, curing procedure, use of special concretes, use of penetrating sealers, use of epoxy-coated steel, etc., can be assessed and their effects on time-to-corrosion and corrosion intensity can be measured in a relatively short time.
2. The chloride ion contents can be measured at the level of the top mat when the macrocell corrosion current begins (i.e., corrosion threshold value), and also at the end of the cyclic testing.

PILOT TIME-TO-CORROSION STUDY ON SELECTED CORROSION PROTECTION SYSTEMS

Testing Program

The corrosion protection systems tested in the pilot studies were selected based upon cost considerations, previous corrosion research, and present utilization in bridge construction. The following variables were evaluated utilizing the macrocell corrosion current measuring techniques.

- Concrete clear cover of 1, 2, and 3 in.
- Target concrete w/c ratios of 0.32, 0.40, and 0.50.
- Use of a high-range water reducing (HRWR) admixture to achieve 0.32 w/c ratio with 3- to 4-in. slump concrete.
- Use of epoxy-coated bars in both mats.
- Use of epoxy-coated bars only in the top mat (potentially anodic region) with ordinary uncoated gray reinforcing bars in the bottom mat (cathodic region).
- Use of galvanized bars in both mats.
- Use of galvanized bars in the top mat with ordinary uncoated gray bars in the bottom mat.
- Use of calcium nitrite "corrosion-inhibiting" admixture in concrete with normal gray bars.
- Use of unstressed normal prestressing strands and normal gray bars in precast construction.
- Use of unstressed epoxy-coated prestressing strands and epoxy-coated bars in precast construction.
- Use of penetrating silane sealer and methacrylate coating systems which have both performed very well in the NCHRP 244(17) accelerated weathering test using the southern climate exposure with ultraviolet light.

These corrosion protection materials were combined with different cover requirements and w/c ratios to produce 58 combinations which could be evaluated under identical conditions.

Ten test combinations, shown in Tables 1 and 2, were used and 124 specimens were tested. Systems 1 and 2 with normal gray bars and epoxy-coated bars, respectively, included all combinations of cover and w/c ratio. Nine pairs of duplicate slabs were tested for each system. Systems 3 to 10 utilized 5 pairs of duplicate slabs for each system.

A special study on ultrasonically-cleaned prestressing strand was included as System 7A, which consisted of four slabs with a w/c ratio of 0.50. Duplicate slabs with 1- and 2-in. cover were tested. System 7 utilized as-received strand, which had a normal coating of drawing lubricants, which could act as a corrosion inhibitor.

Table 1 - Test program layout for Systems 1 and 2.

Concrete cover, in.	No. of concrete slabs		
	0.50	0.40	0.32, w/c ratio
1	2	2	2
2	2	2	2
3	2	2	2

System No.	Main variable	Purpose of tests	Total No. of slabs
1	All normal reinforcing bars	To establish the performance of normal concrete construction when using normal reinforcing bars to provide a baseline for evaluating the other corrosion protection systems.	18
2	All reinforcing bars epoxy coated	To establish the performance of epoxy-coated bars when used in both layers.	18

Table 2 - Test program layout for Systems 3 to 10.

Concrete cover, in.	No. of concrete slabs 0.50	 0.40	 0.32, w/c ratio
1	2		
2	2	2	2
3	2		

System No.	Main variable	Purpose of tests	Total No. of slabs
3	Epoxy-coated reinforcing bar and normal bars used in same construction	To establish the difference in corrosion resistance when epoxy-coated bars are used in the top layer and normal bars in the bottom layer. This method is being used by numerous highway agencies.	10
4	Galvanized bars used throughout concrete	To establish the performance of galvanized bars when used in both layers.	10
5	Galvanized bars and normal bars used in same construction	To establish the difference in corrosion resistance when galvanized bars are used in the top layer and normal bars in the bottom layer.	10
6	Calcium nitrite admixture as a corrosion inhibitor in concrete	To establish the corrosion-inhibiting capability of calcium nitrite admixture when added to fresh concrete. This admixture is currently being used by several highway agencies.	10
7	Normal prestressing strands and normal reinforcing bars in precast construction	To establish the corrosion resistance of normal prestressing strands in the top layer with normal gray bars in the bottom layer.	10
8	Epoxy-coated prestressing strands and epoxy-coated bars in precast construction	To establish the corrosion resistance of precast bridge decks (stay-in-place forms) and new replacement decks when using epoxy-coated strands in the top layer and epoxy-coated bars in the bottom layer.	10
9	Penetrating sealer on normal concrete construction	To establish the corrosion performance of penetrating sealer when used on normal concrete with gray bars. This silane sealer performed well in the NCHRP 244 accelerated southern climate weathering tests.	10
10	Coating on normal concrete construction	To establish the corrosion performance of a coating material when used on normal concrete construction with gray bars. This methyl methacrylate-ethyl acrylate coating system performed well in NCHRP 244 accelerated southern climate weathering tests.	10

Test Specimen Design and Test Details

Fig. 2 shows the 12-in. square slabs of variable depths which were tested with clear covers of 1, 2 and 3 in. The slab thicknesses were 7, 8 and 9 in., depending upon cover. A constant 4-in. thickness of concrete was therefore maintained between all top and bottom bars. The bottom layer of No. 4 bars had 1-in. clear cover to provide these bars equal access to atmospheric oxygen in all specimens. These four bottom bars provided a cathodic region due to the use of initially chloride-free concrete. The top bars were joined into a top steel mat with an external buss bar which was electrically connected, by a 10-ohm resistor, to the buss bar joining the bottom reinforcing steel mat. The voltage drop across the 10-ohm resistor provided a means for calculating corrosion currents.

All slabs were moist cured for 3 days, to simulate field-curing conditions. They were then exposed to at least 25 days of air drying at 60 to 80°F prior to starting the accelerated testing. Ponded saltwater was a 15 percent sodium chloride solution which contains about 9.1 percent chloride ions by weight of water. This chloride content is about 5 times greater than that in typical seawater. The weekly test cycle was as follows:

- One hundred hours with salt water on top surface at 60 to 80°F, followed by vacuum removal of saltwater.
- A fresh water rinse, followed by vacuum removal of fresh water.
- Sixty-eight hours drying at 100°F.

This weekly cycle was repeated 48 times. Fig. 3 shows the test area in which half the slabs are being ponded while the other half are heated to 100°F under canopies. Once during each week, the corrosion current was measured. Copper-copper sulfate half-cell tests were made on a monthly basis. Also each month, I.O.P. and concrete AC electrical resistance were measured between the top and bottom steel mats. The measurement methods are described in Appendix A.

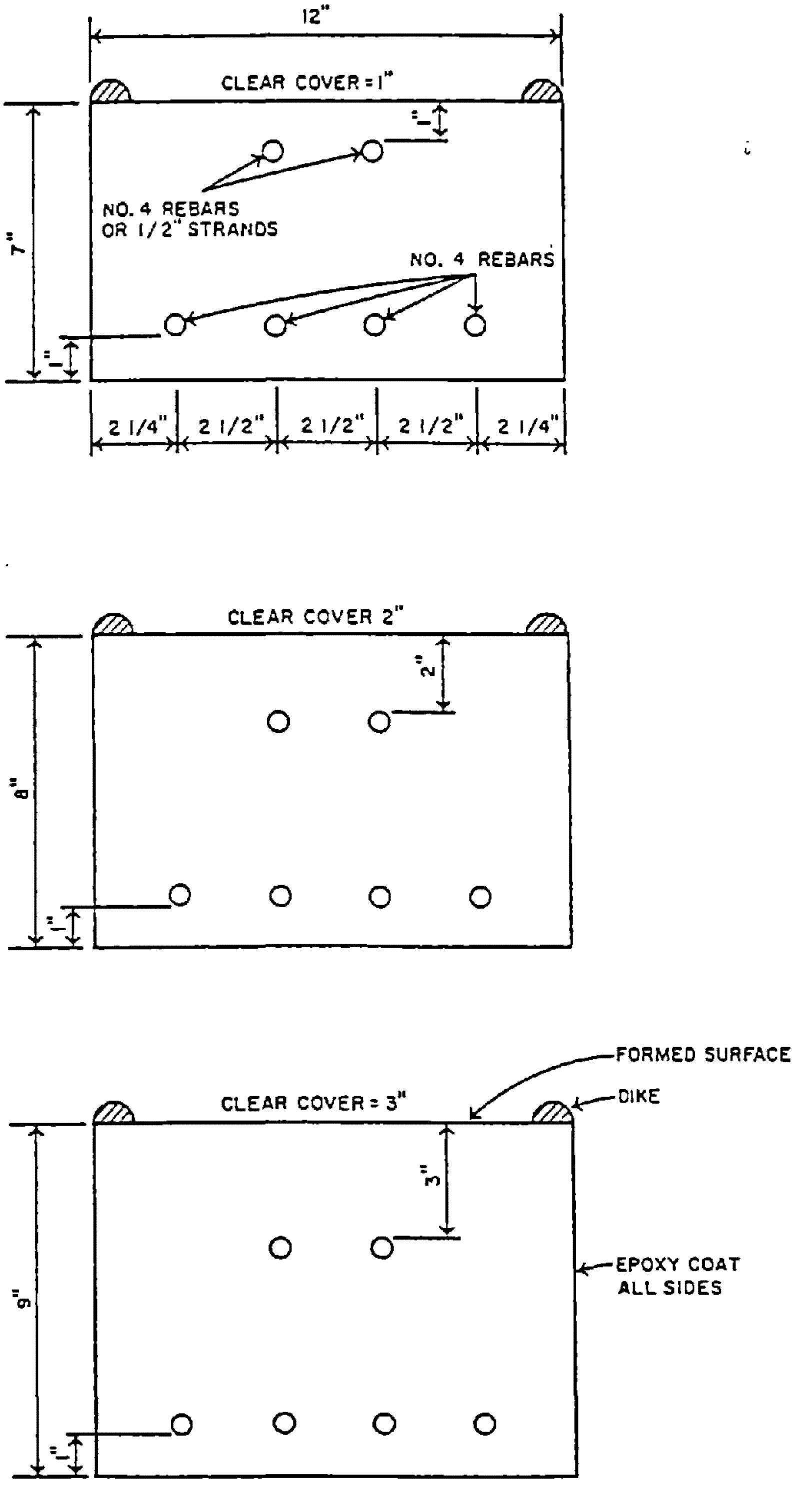


Fig. 2 - Cross section of Pilot Study time-to-corrosion specimens.

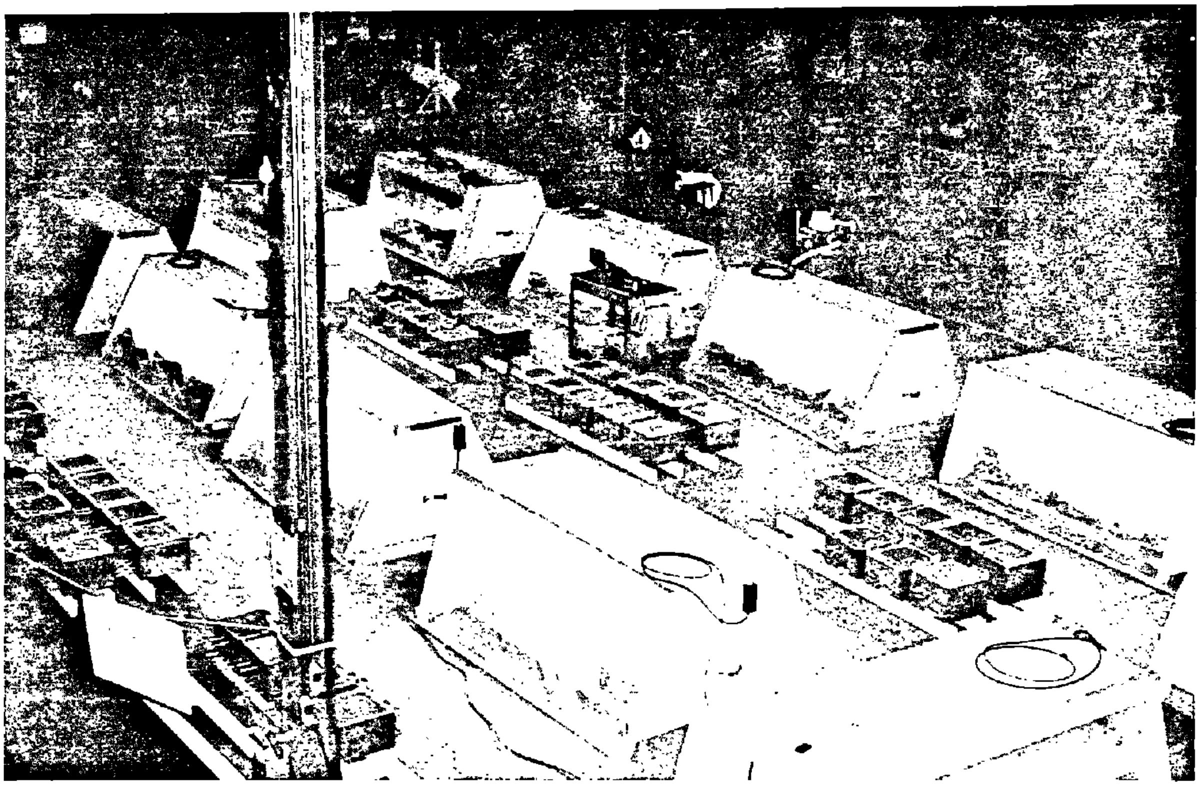


Fig. 3 - Pilot time-to-corrosion study cyclic test area.

Chloride ion content tests (acid-soluble) were made when the initial steady corrosion current flow indicated the start of macrocell corrosion. Powder samples were obtained by drilling three 1/4-in. holes each into the two sides of the slabs in the plane of the top surface of the top bars. The hole depth was 2 in. with the powder sample portion as shown in Fig. 4. The holes were then filled with a sanded epoxy. Chloride contents were also determined near the conclusion of the 48-week period.

Specific Details

Further details concerning the reinforcing bars and strands, the concrete mixtures and materials, the specimen fabrication details, chloride ion content tests and coating procedures are provided in Appendix B.

Tests Results and Discussion

The test results will be discussed in the following sections. They will include time-to-initiation of corrosion, chloride content at initiation of corrosion, corrosion activity characteristics of different systems and mat-to-mat electrical resistance properties. They will also include visual examination of bars and strands after the cyclic tests and chloride profiles after 44 weeks of cyclic testing.

Time-to-Corrosion

The measured data from the 22 specimens that developed significant corrosion activity during the cyclic testing are tabulated in Table 3. The other 102 specimens did not develop corrosion activity. The data listed are number of cycles to time-to-corrosion; corrosion current, I.O.P., and half-cell potential after 11, 22, 33 and 44 weeks of cyclic testing; chloride ion content at the level of the top surface of the top bars at time-to-corrosion and after 44 weeks. Also measured was AC electrical resistance between the two mats of steel at the beginning of the tests and after 44 weeks. Plots of actual measured corrosion current,

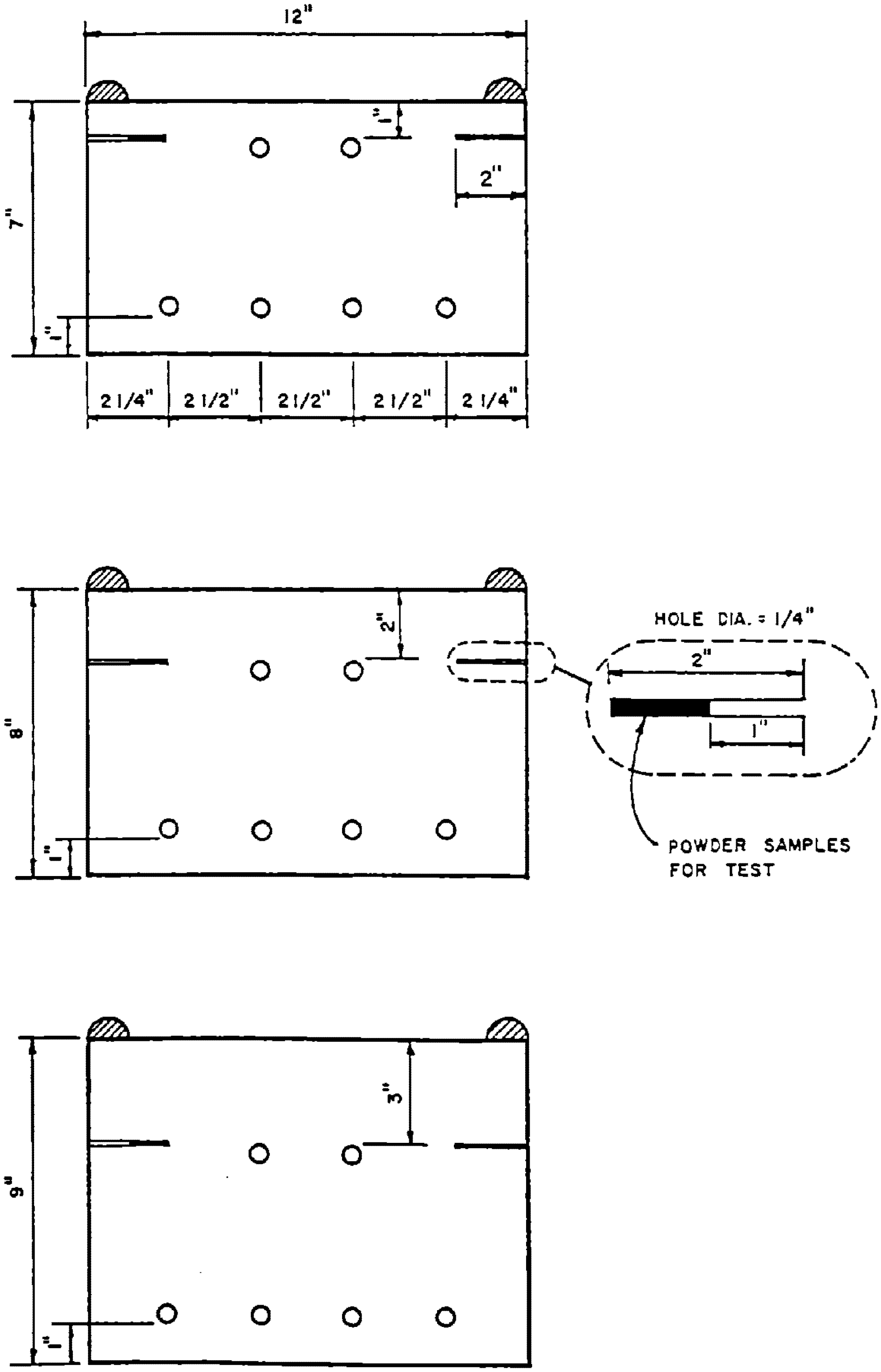


Fig. 4 - Location for drilled powder samples to measure chloride content at initiation of corrosion and after 44 weeks.

Table 3 - Summary of corrosion activity data for pilot time-to-corrosion study specimens.

Specimen No. (System-Cover-W/C)	Time to corrosion, weeks	Corrosion current, μA				Instant-off potential, mV				Half-cell potential, V				Cl^- content, % by wt. of concrete		Mat-to-mat AC resistance, ohms	
		11 Weeks	22 Weeks	33 Weeks	44 Weeks	11 Weeks	22 Weeks	33 Weeks	44 Weeks	11 Weeks	22 Weeks	33 Weeks	44 Weeks	Time to Corrosion	44 Weeks	Initial	44 Weeks
1-1-50-A	8	90	220	300	260	23	79	108	118	-.349	-.459	-.542	-.557	N/A	0.443	185	450
1-1-50-B	5	90	120	180	180	29	61	102	122	-.347	-.412	-.470	-.510	0.023	0.407	190	660
1-1-40-A	4	70	70	65	70	17	31	34	50	-.312	-.361	-.334	-.377	0.022	0.079	240	890
1-1-40-B	6	68	19	12.5	60	12	9	7	51	-.282	-.260	-.274	-.342	0.049	0.046	225	940
1-1-32-A	3	45	90	155	110	18	38	76	76	-.308	-.369	-.436	-.445	0.024	0.030	265	890
1-1-32-B	2	15	11	10	2	6	14	4	3	-.281	-.246	-.225	-.216	0.048	0.027	250	920
1A-1-40-A	12	30	19	N/A	N/A	12	12	N/A	N/A	-.273	-.256	N/A	N/A	0.027	N/A	230	N/A
1A-1-40-B	16	0	90	N/A	N/A	0	45	N/A	N/A	-.116	-.368	N/A	N/A	0.043	N/A	235	N/A
1A-1-32-A	4	35	2	N/A	N/A	15	2	N/A	N/A	-.327	-.248	N/A	N/A	0.018	N/A	245	N/A
5-1-50-A	9	21	170	210	100	8	96	186	154	-.254	-.394	-.602	-.514	0.056	0.396	200	1900
5-1-50-B	11	29	280	130	41	12	184	121	109	-.275	-.557	-.504	-.452	N/A	0.582	190	2400
6-1-50-A	10	26	70	120	N/A	7	21	69	N/A	-.252	-.340	-.391	N/A	0.016	0.252	160	560
6-1-50-B	9	32	31	15	N/A	9	12	10	N/A	-.275	-.304	-.258	N/A	0.017	N/A	170	630
7-1-50-A	28	0	0	105	50	0	0	50	51	-.161	-.166	-.379	-.413	0.180	0.498	200	1010
7-1-50-B	11	25	110	167	110	15	44	91	101	-.327	-.369	-.453	-.475	0.081	0.488	180	1100
7A-1-50-A	15	1	90	110	80	0	38	60	71	-.179	-.375	-.413	-.418	0.102	0.482	220	1000
7A-1-50-B	24	1	4	80	44	0	3	56	65	-.166	-.237	-.397	-.396	0.197	0.371	210	1350
9-1-50-A	29	0	0	20	13	0	0	18	17	-.048	-.060	-.230	-.210	N/A	0.021	210	1500
10-1-50-A	13	0	250	300	190	0	122	139	147	-.170	-.512	-.557	-.604	0.018	0.670	200	790
10-1-50-B	32	0	0	37	80	0	0	48	152	-.103	-.097	-.282	-.467	0.117	0.219	190	1950
10-2-50-A	44	0	0	0	36	0	0	0	48	-.154	-.117	-.127	-.326	0.009	0.009	180	1300
10-2-50-B	44	0	0	0	44	0	0	0	51	-.144	-.122	-.143	-.357	0.022	0.022	170	1100

I.O.P., and half-cell potentials versus time under test are given in Figs. 32 to 42 in Appendix C.

Normal gray bars - The System 1 slabs with 1-in. cover and nominal w/c ratios of 0.50, 0.40, and 0.32 all showed corrosion activity after as little as 2 to 8 weeks of cyclic testing. At the start of corrosion activity, each of these 6 slabs exhibited significant and simultaneous increases in corrosion current, I.O.P, and half-cell potentials. Since the 1-in. cover, 0.40 and 0.32 w/c ratio slabs showed surprisingly early corrosion, specimens with these two test conditions were recast and retested. After 4 weeks of retesting, one of the duplicate 0.32 w/c ratio slabs exhibited corrosion activity, and the two duplicated slabs with the 0.40 w/c ratio showed corrosion activity after 12 and 16 weeks. The average time-to-corrosion period for these 9 specimens was as follows.

Nominal w/c	Number of specimens corroding	Average time-to-corrosion, weeks
0.50	2	6.5
0.40	4	9.5
0.32	3	3.0

During a concurrent private industry-sponsored study, 50 slabs were tested using the 0.50 w/c ratio and 1-in. cover condition with normal gray bars. The average time-to-corrosion for these 50 identical slabs was 5.9 weeks. This compares well with the average of 6.5 weeks for the two comparable slabs from System 1. These data show that with 1 in. of cover, the w/c ratio difference did not provide significant and consistent differences in time-to-corrosion.

None of the slabs with 2- or 3-in. cover, with any of the w/c ratios, exhibited any signs of corrosion activity during the entire test period.

These results indicate that the amount of clear cover is extremely important. Also, 1-in. clear cover, irrespective of w/c ratio, is not adequate protection against chloride-induced corrosion of gray bars.

Epoxy-coated bars used in both mats - None of the System 2 slabs showed any detectable corrosion activity.

Epoxy-coated bars and gray bars used in same construction - None of the System 3 slabs showed any detectable corrosion activity.

Galvanized bars used in both mats - None of the System 4 slabs exhibited any significant or sustained evidence of zinc corrosion current activity. However, as shown in Appendix C, low corrosion currents were measured throughout the testing and significant surges in half-cell potentials were observed at about the time period when the chloride ion should have reached threshold concentration next to the galvanized bars at the 1-in. level in the 0.50 w/c ratio slabs.

Galvanized bars and gray bars used in same construction - Both System 5 slabs with 1-in. cover and the 0.50 w/c ratio exhibited evidence of zinc corrosion starting at an average age of 10 weeks. This time-to-corrosion is similar to that determined for the System 1 slabs with gray bars for the same test conditions. None of the slabs with 2- or 3-in. cover, with any w/c ratio, exhibited any evidence of zinc corrosion.

Calcium nitrite as an admixture - Both System 6 slabs with 1-in. cover and the 0.50 w/c ratio showed corrosion activity at an average age of 9.5 weeks. This time-to-corrosion is similar to the System 1 and 5 slabs as previously discussed. None of the slabs with 2- or 3-in. cover, with any w/c ratio, exhibited detectable corrosion.

Normal prestressing strands and normal gray bars used in same construction - The four System 7 and 7A slabs with as-received strands and ultrasonically-cleaned strands, with 1-in. cover and 0.50 w/c ratio, exhibited signs of corrosion at average ages of 19.0 and 19.5 weeks, respectively. None of the slabs with 2- or 3-in. cover, with any w/c ratio, had measurable corrosion. These data show that the ultrasonic cleaning of the as-received strands did not influence the time-to-corrosion. However, the average time-to-corrosion with the prestressing strands was almost 3 times as long as with normal gray bars.

Epoxy-coated strands and epoxy-coated bars used in same construction - None of the System 8 slabs showed any detectable corrosion.

Penetrating silane sealer on concrete - One of the System 9 slabs with the 0.50 w/c ratio and 1-in. cover exhibited minor corrosion activity at 29 weeks. All other specimens showed no evidence of corrosion activity.

Coating on concrete - Both of the System 10 slabs with the 0.50 w/c ratio with the 1- and 2-in. cover exhibited corrosion activity. The slabs with 1-in. cover showed corrosion activity at an average age of 22.5 weeks while the average time-to-corrosion for the 2-in. cover condition was 44 weeks. These specimens were the only ones in the entire program that had 2-in. cover and exhibited corrosion activity.

Chloride Content at Initiation of Corrosion

Normal gray bars - The average chloride content at time-to-corrosion for the seven tested System 1 slabs with 1-in. cover and nominal 0.50, 0.40, and 0.32 w/c ratios was 0.032 percent by weight of concrete. The range was from 0.018 to 0.049 percent. This average is equivalent to 1.26 lbs of acid-soluble chloride ion per cu yd of concrete. The average corrosion threshold chloride ion content for the three w/c ratios were as follows:

Actual w/c	Chloride ion, % by wt. of concrete
0.51	0.023
0.40	0.038
0.28	0.030

These three chloride ion values were equivalent to 0.91, 1.50 and 1.19 lbs of acid-soluble chloride ion per cu yd, which are the same as 0.21, 0.26 and 0.17 percent chloride ion by weight of portland cement, respectively. These average chloride ion values correlate well with previous threshold values reported by Lewis(3) and Clear.(8,12)

In the concurrent private industry study, 48 slabs which utilized 1-in. cover, gray bars and a w/c ratio of 0.50 indicated an average chloride ion corrosion threshold of 0.024 percent by weight of concrete. This is equivalent to 0.219 percent by weight of cement. This average from 48 identical slabs is essentially equal to the value measured on the one System 1 slab (0.023 percent by weight of concrete) previously discussed.

The powder drilling technique used a 1/4-in. drill bit to minimize errors due to steep chloride gradients. Because of the small size of sample thus obtained, a supplementary test series was undertaken to evaluate the chloride ion test technique. A concrete mixture using the same 0.50 w/c ratio concrete was produced with five known amounts of admixed chloride ion. These were 0.0098, 0.0188, 0.0289, 0.0409 and 0.0554 percent chloride ion by weight of concrete. Six cylinders (6 x 12 in.) were cast and moist cured. Two cylinders were from the lowest chloride ion content concrete. One cylinder each was made from the four higher values. The powder drilling technique was as follows:

- Three 1/4-in. diameter holes were drilled into opposite sides of each cylinder.
- The first inch of powder was discarded.
- The sample to be tested consisted of the powder from the 1- to 2 1/2-in. depth increment.
- The powder from each three-hole group was combined into one sample for testing.

The test results from these 12 powder samples are shown in Fig. 31 in Appendix B. The average values show a good relationship between "as-mixed" and "as-tested" values and indicate good accuracy for the method of sampling and testing.

Galvanized bars - The chloride ion content of the one tested System 5 slab was 0.056 percent by weight of concrete at time of zinc corrosion.

Calcium nitrite and gray bars - The average threshold chloride ion content for the System 6 slabs was 0.0165 percent by weight of concrete, which is equivalent to 0.65 lbs/cu yd or 0.15 percent by weight of portland cement.

Normal prestressing strands - The average threshold chloride ion contents for the slabs with as-received and ultrasonically-cleaned strands were 0.13 and 0.15 percent by weight of concrete, respectively. The overall average for these four slabs was 0.14 percent. The data show that the ultrasonic cleaning had no effect on average chloride ion content at time-to-corrosion. The data also show that the (unstressed) prestressing strands tolerated 6 times as much chloride ion as normal reinforcing bars. The average time-to-corrosion was about 3 times longer when they were embedded in a 0.50 w/c ratio concrete and provided 1-in. cover.

Methacrylate coating on concrete - The two System 10 slabs with 1-in. cover had threshold chloride values of 0.018 and 0.117 percent by weight of concrete. Such large differences suggest that the coating provided non-uniform protection. The two slabs with 2-in. cover had corrosion threshold values of 0.009 and 0.022 percent, both at 44 weeks. Three of the four threshold chloride ion values were rather uniform. Their average value was 0.016 percent, somewhat lower than the System 1 value of 0.023 percent.

Corrosion Activity Characteristics of Different Systems

The corrosion measurements were made on damp concrete within temperature limits of 60 to 80°F. The measured data in Appendix C reflect the temperature variations within this range.

Normal gray bars - The average half-cell data from the System 1 slabs, with the 0.50 w/c ratio and 1-, 2- and 3-in. cover, are shown in Fig. 5. With 1-in. cover, the half-cell potential showed an increase at early ages and continued to increase throughout the testing period. The specimens with 2- and 3-in. cover showed no increase in half-cell potential during the entire testing period. These results show the very significant effect that 2- and 3-in. cover had on preventing corrosion activity.

The specimens with 0.40 and 0.32 w/c ratios with 1-in. cover also showed similar early-age increases in half-cell potentials. Specimens with 2- and 3-in. cover, at all w/c ratios, exhibited no detectable corrosion currents or change in half-cell potentials at any time.

After initiation of corrosion, the 0.50 w/c ratio concrete with 1-in. cover exhibited continually increasing corrosion currents up to about 30 weeks. From then on, a small decline in current was measured. The specimens with the 0.40 and 0.32 w/c ratios with 1-in. cover exhibited much lower corrosion currents during the cyclic testing compared to the 0.50 w/c ratio concrete.

The simultaneous increases in corrosion current, I.O.P., and half-cell potentials showed that corrosion was beginning at low half-cell potentials compared to the guide lines in ASTM C876.(23) The corrosion current data versus half-cell potentials were plotted for the System 1 slabs with 1-in. cover and 0.50 w/c ratio. These data show that corrosion currents begin when the half-cell potential exceeds about -0.20 to -0.25 volts. There is also a nearly linear relationship between corrosion current and half-cell

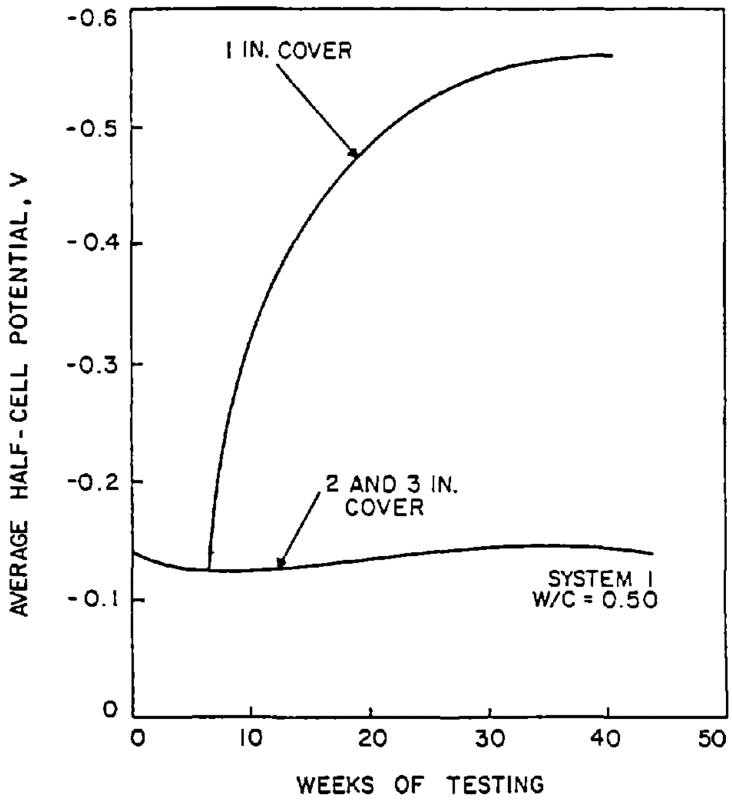


Fig. 5 - Average half-cell potentials for System 1 specimens with 0.50 w/c ratio.

potential as corrosion intensifies. Similar linear trends were found in the relationship between I.O.P. and half-cell potential.

The corrosion current versus half-cell potential data have been plotted for the two slabs previously discussed together with the 50 slabs from the private industry study having the same 0.50 w/c ratio and 1-in. cover conditions. Data relating 209 different half-cell readings when active corrosion currents were measurable from these 52 slabs are shown in Fig. 6. These data show a reasonably linear relationship between corrosion current and half-cell potential. A linear regression analysis was made. The resulting equation is as follows:

$$I_c = -774.2P - 184.2$$

where I_c = corrosion current, microamperes
P = half-cell potential, volts

This analysis produced a correlation coefficient of 0.920. The equation indicates that the corrosion activity starts at a half-cell potential of about -0.24 volts. Similar plots of corrosion current versus half-cell potential, made from the System 6 slabs with calcium nitrite, exhibit a similar linear trend. The data show that active corrosion occurred at half-cell potentials as low as -0.20 to -0.25 volts and that significant corrosion current developed in the "uncertain range," i.e., -0.20 to -0.35 volts, as discussed in ASTM C876.(23)

Galvanized bars - The specimens from both Systems 4 and 5 exhibit immediate but very small zinc corrosion currents prior to the time that chloride ion could have penetrated to the bars, during the first several weeks of cyclic testing.

The System 4 slabs with galvanized bars in both mats show high half-cell potentials of about -0.65 volts at the start of the testing. A surge

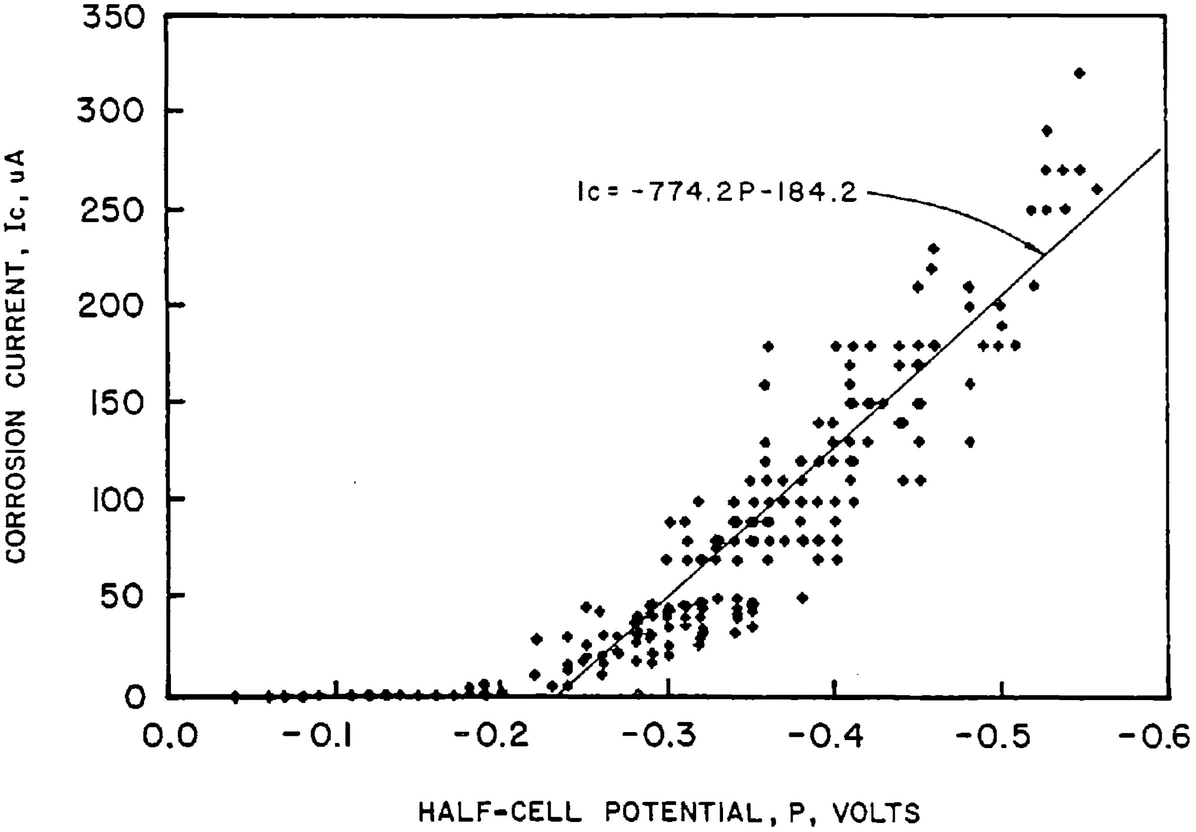


Fig. 6 - Relationship between corrosion current and half-cell potential for 52 identical System 1 specimens with 1-in. cover and a 0.50 w/c ratio.

in half-cell potential developed at age 9 weeks. The half-cell potential eventually reached about -0.8 volts. At about 30 weeks the half-cell potential began to decline rapidly. These specimens showed only very low corrosion currents during the entire cyclic testing.

The System 5 specimens, with galvanized bars in the top mat and gray bars in the bottom mat, behaved differently when compared to the System 4 specimens. The half-cell data was similar to that of comparable System 1 specimens. The development of high corrosion currents was also similar to the System 1 slabs except for the drop in corrosion current and half-cell potential subsidence after about 25 to 30 weeks of testing.

Calcium nitrite admixture - The System 6 specimens with 1-in. cover and 0.50 w/c ratio exhibited corrosion activity at about 10 weeks. These data show that the time-to-corrosion for these calcium nitrite specimens was not significantly delayed compared to the conventional concrete specimens. The ratio of chloride ion to nitrite ion was 0.073 at the time-to-corrosion. While the nitrite ion (8.88 lbs/cu yd) did not totally prevent corrosion, the amount of subsequent corrosion current was significantly reduced compared to conventional concrete.

Normal prestressing strands - The as-received and ultrasonically-cleaned strands each developed similar amounts of corrosion current. The corrosion currents were less than that developed with conventional gray bars. Both types of strands also exhibited longer time-to-corrosion than gray bar.

Mat-to-Mat Electrical Resistance Properties

The initial mat-to-mat AC electrical resistance values of the specimens which corroded, and another reading after 44 weeks are listed in Table 3. The review of the AC resistance data from all 124 specimens has led to the following conclusions:

- As shown in Fig. 7, the mat-to-mat resistance and concrete resistivity of all 7-, 8- and 9-in. thick specimens with gray bars and a 0.50 w/c ratio concrete were within a small range at any given age. These System 1 conventional concrete specimens had a concrete resistivity of about 10,000 ohm-cm at age 28 days, when the cyclic tests started. The corrosion current data in Appendix C shows that corrosion current for the conventional concretes with gray bar or strand increased rather uniformly until the resistivity was about 25,000 to 30,000 ohm-cm. At these high levels of resistance, the corrosion currents either stabilized or declined slightly. Resistivity values eventually reached about 35,000 to 45,000 ohm-cm at age 48 weeks.

- The gray prestressing strand specimens (System 7) exhibited the same AC resistance as those containing only gray bars.

- As shown in Fig. 7, silane-treated conventional concretes developed much higher resistivity values as the cyclic testing progressed. This is attributed to a decrease in water absorption into these 7-, 8- or 9-in. thick slabs during the cyclic testing.

- The System 6 specimens made with calcium nitrite exhibited the same concrete resistivity as conventional concrete specimens.

- The mat-to-mat AC resistance of the galvanized bar specimens (Systems 4 and 5) were equal to that of gray bar specimens at the start of the cyclic testing. As the testing progressed and zinc corrosion products developed, the mat-to-mat electrical resistance values increased to 2,000 to 2,600 ohms at the 48-week period. These final AC resistance values are 2 to 4 times as high as comparable System 1 gray bar specimen values at 48 weeks.

- The mat-to-mat AC resistances of the specimens with epoxy-coated bars (Systems 2 and 3) were very high and essentially in the same range. Thus, the use of epoxy-coated bars in both mats did not influence the resistance values as compared to the System 3 specimens with gray bars in the bottom mat. The values ranged from 10,000 to 230,000 ohms at the start of testing with the System 2 specimens. The System 3 specimens initial values ranged from 2,000 to 230,000 ohms. The 48-week values ranged from 65,000 to 820,000 and from 20,000 to 710,000 ohms for System 2 and 3 specimens, respectively. This wide range in resistance values was caused by different, yet acceptable, holiday defects in the coatings. This wide range in values was not related to specimen thickness, concrete w/c ratio, or clear cover.

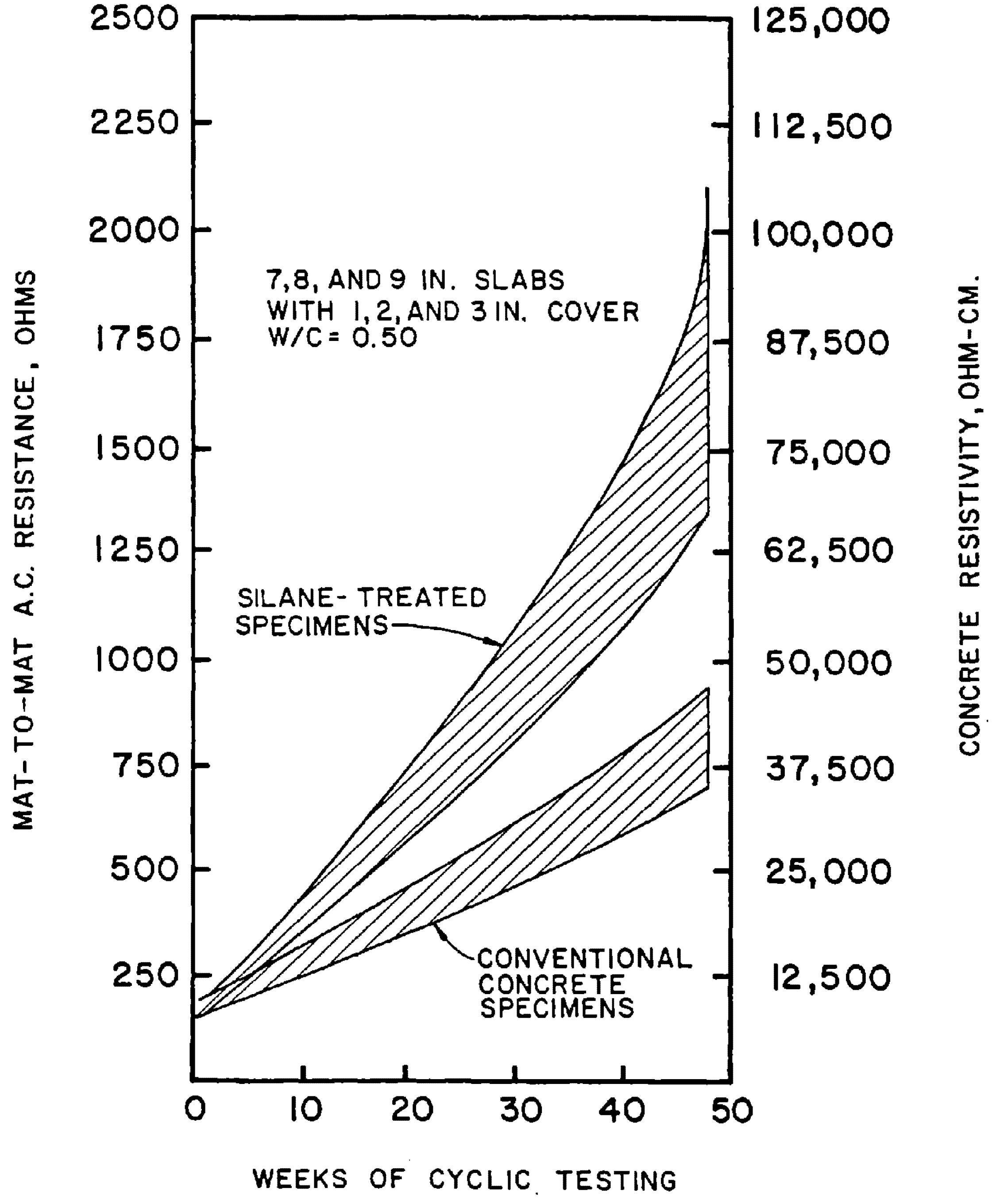


Fig. 7 - Mat-to-mat resistance and concrete resistivity of System 1 and 9 specimens.

- The mat-to-mat AC resistance of the epoxy-coated strand specimens (System 8) was greater than 1 megohm during the entire 48-week test period. This extremely high value is indicative of an electrical insulator with no holidays.
- The final mat-to-mat AC resistance values from the System 10 methacrylate-coated specimens showed wide variation, which suggests coating deterioration problems.

Visual Examination of Bars and Strands After Cyclic Tests

The corroded specimens, generally with 1-in. cover, were split at the end of the cyclic testing. The bars or strands were examined visually for degree of corrosion and then with a 15 to 75 power stereo microscope to better define the degree of corrosion, such as the extent of pitting. Each bar or strand was measured to estimate the percent of the surface area on the top-half perimeter which was corroded. In all cases, the bottom half of the bar perimeters in the top mat were found to be essentially noncorroded. All of the bars in the bottom mat were found to be free of corrosion. The results of this inspection are summarized in Table 4 and discussed below.

Normal gray bars - The System 1 slabs with 0.50, 0.40, and 0.32 w/c ratios had corrosion products on approximately 73, 9, and 11 percent of the upper perimeter, respectively. The corrosion was particularly severe on and adjacent to the bar deformations. Pitting was sometimes observed in this region.

Epoxy-coated bars used in both mats - The epoxy-coated bars in the System 2 slab with the 0.50 w/c ratio exhibited no visible corrosion products. Even the original holiday locations, which had been previously marked with ink, were clean.

Epoxy-coated bars and gray bars used in same construction - The epoxy-coated bars in the System 3 slab, with the 0.50 w/c ratio, exhibited no

Table 4 - Summary of visual corrosion data for reinforcing steel in the top mat of tested systems.

System No.	Cover, in.	Nominal w/c	Slab No.	Estimated percent of surface corroded on top-half perimeter of bar		Comments
				Ind	Avg	
1	1	0.50	A	75	73	Severe corrosion on and near deformations
1	1	0.50	A	100		Severe corrosion on and near deformations
1	1	0.50	B	80		Severe corrosion on and near deformations
1	1	0.50	B	35		Severe corrosion on and near deformations
1	1	0.40	A	0.1	9	Two small rust spots
1	1	0.40	A	20		
1	1	0.40	B	0.1		Two small rust spots
1	1	0.40	B	15		
1	1	0.32	A	40	11	Severe corrosion on and near deformations
1	1	0.32	A	0		
1	1	0.32	B	4		
1	1	0.32	B	0		
2	1	0.50	A	0	0	No rust by 'holidays' previously marked
2	1	0.50	A	0		Very little cementitious bond to bar
3	1	0.50	A	0	0	No rust by 'holidays' previously marked
3	1	0.50	A	0		Very little cementitious bond to bar
4	1	0.50	B	0	0	No rust
4	1	0.50	B	0		No rust
5	1	0.50	A	0.1	1.5	Two small rust spots
5	1	0.50	A	0.1		Three small rust spots
5	1	0.50	B	2		Twelve small rust spots
5	1	0.50	B	4		Twenty small rust spots
6	1	0.50	A	0.1	7.5	One rust spot
6	1	0.50	A	20		
6	1	0.50	B	0.1		One rust spot
6	1	0.50	B	10		
7	1	0.50	A	0	16	Little cementitious bond to strand
7	1	0.50	A	25		Deep localized pitting
7	1	0.50	B	3		One rust spot
7	1	0.50	B	35		Deep localized pitting
7A	1	0.50	A	10	8	Better cementitious bond to strand Deep localized pitting
7A	1	0.50	A	10		Better cementitious bond to strand Deep localized pitting
7A	1	0.50	B	5		Better cementitious bond to strand Deep localized pitting
7A	1	0.50	B	7		Better cementitious bond to strand Deep localized pitting
8	1	0.50	A	0	0	Good cementitious bond to epoxy coating
8	1	0.50	A	0		Good cementitious bond to epoxy coating
9	1	0.50	A	9	4.5	
9	1	0.50	A	0		
10	1	0.50	A	40	34	Severe corrosion
10	1	0.50	A	45		Severe corrosion
10	1	0.50	B	0		
10	1	0.50	B	50		Severe corrosion
10	2	0.50	A	0	7.5	
10	2	0.50	A	15		
10	2	0.50	B	0		
10	2	0.30	B	15		

visible corrosion products. The original holiday locations, which had been previously marked with ink, again were clean.

Galvanized bars used in both mats - The galvanized bars in the System 4 slab with the 0.50 w/c ratio had no visible steel corrosion by-products. Zinc corrosion products, present as whitish and blackish gray discoloration, covered most of the top half of the bar perimeter. Scrapings from these areas were analyzed by X-ray diffraction methods. They were identified as zinc oxide (ZnO) and zinc hydroxychloride [$(ZnCl_2 \cdot 4Zn(OH)_2$], respectively.

Galvanized bar and gray bars used in same construction - The top half of the galvanized bar perimeter from the two System 5 slabs, with the 0.50 w/c ratio, had localized steel corrosion pitting. Red rust was evident on approximately 1.5 percent of the galvanized surface. These localized red rust pits were nominally 0.10 in. in diameter. Zinc corrosion by-products, whitish and blackish discoloration of the zinc, were found along the bar length. These zinc by-products covered almost the entire bar perimeter.

Calcium nitrite as an admixture - The gray bars in the System 6 slabs, with 1-in. cover and 0.50 w/c ratio, had corrosion on approximately 8 percent of the upper perimeter surface.

Normal prestressing strands and normal bars used in same construction - The top-half perimeter of the as-received and ultrasonically-cleaned strands from the two System 7 and 7A slabs, with the 0.50 w/c ratio, showed corrosion on approximately 16 and 8 percent of the surface, respectively. Both strand types had localized corrosion pitting within these areas.

Epoxy-coated strands and epoxy-coated bars in same construction - The epoxy-coated strands in the System 8 slab with the 0.50 w/c ratio exhibited no visible corrosion products. The sanded epoxy surface provided good cementitious bond of the concrete to the epoxy-coated strands.

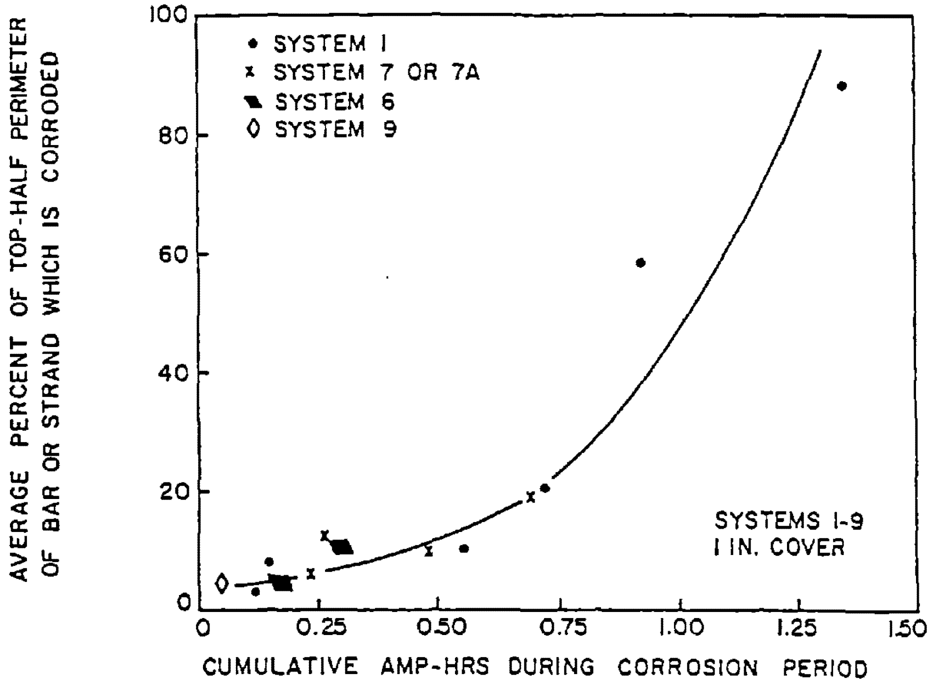
Penetrating silane sealer on concrete - One silane-sealed specimen with a 0.50 w/c ratio from System 9 developed minor corrosion activity at 29 weeks. Approximately 5 percent of the bar surfaces were corroded. The companion specimen was free of corrosion.

Coating on concrete - The methacrylate-coated slabs from System 10 with the 0.50 w/c ratio with 1- and 2-in. cover had approximately 34 and 8 percent corrosion of the bar surfaces, respectively.

General discussion - Inspection of 44 bars or strands from 22 slabs showed that rust was present on all bars or strand that had measurable corrosion current activity. The System 5 galvanized bars developed significant zinc corrosion current activity. Those specimens which showed no significant corrosion activity during the testing were found to be free of steel corrosion by-products. This included the System 4 slabs with galvanized bars in both mats. Thus, the visual inspection substantiated the electrical data measurements made during the cyclic testing.

These tests on ten corrosion protection systems resulted in widely different corrosion protection with 1-in. cover and 0.50 w/c ratio concrete. The average percentage of the top-half bar perimeter area which was corroded ranged from 0 to 88 percent.

Because these tests were made under relatively constant temperature conditions, plots of corrosion current versus time provided a means of estimating the cumulative amp-hours of corrosion current developed. These amp-hour values were determined by integrating the area under a smoothed curve of corrosion current versus time. These data are shown tabulated as well as plotted in Fig. 8 against average percent of the top-half perimeter which was corroded in the Systems 1 to 9 specimens. The



System No.	Slab No.	Cover (in.)	Nominal w/c	Avg. percent surface area corroded*	Cumulative amp-hrs.**
1	A	1	0.50	88	1.34
1	B	1	0.50	58	0.92
1	A	1	0.40	10	0.35
1	B	1	0.40	8	0.15
1	A	1	0.32	20	0.72
1	B	1	0.32	2	0.12
6	A	1	0.50	10	0.29***
6	B	1	0.50	5	0.17
7	A	1	0.50	13	0.27
7	B	1	0.50	19	0.70
7A	A	1	0.50	10	0.49
7A	B	1	0.50	6	0.23
9	A	1	0.50	4	0.04
10	A	1	0.50	43	1.31
10	B	1	0.50	25	0.11

* Of top half perimeter from both top bars or strands
** From time-of-corrosion to 44 weeks
*** From time-of-corrosion to 33 weeks

Fig. 8 - Relationship between percent of surfaces of bars or strands corroded and cumulative amp-hrs during corrosion period.

System 10 data have been omitted due to the unusual data obtained, which may relate to non-uniform protection.

The cumulative amp-hour data developed from macrocell corrosion current ranged from 0.04 to 1.34 amp-hours. The literature(24) indicates that 1.0 amp-hour of corrosion current consumes 1.04 grams of iron. Thus, these data indicate iron consumption values of only 0.04 to 1.40 grams. These values represent very low percentages of the nominal 435 to 555 gram weights for the two 11 in. pieces of strand or bar used in the anodic region. In the worst case (Slab A of System 1 with the 0.50 w/c and 1-in. cover), the average uniform steel thickness loss calculated to be only 0.0007 in. over the corroded region, with 1.34 amp-hours of current and 88 percent of the bar half perimeter corroded. Similar calculations for the other System 1, 6, 9 and 10 slabs which utilized gray bar resulted in average steel thickness losses of 0.0003 to 0.0025 in. over the corroded region. While these losses are low, extensive rusting was evident on gray bars, pitting was evident on bare prestressing strand, and cracking of the concrete and surface rust deposits were apparent on some concrete slabs. The bottom-half perimeter of each bar or strand was noncorroded and areas along the top-half perimeter were usually noncorroded. Therefore it is probable that strong microcell corrosion developed within the bar or strand and were not measured during these tests, which measured only mat-to-mat macrocell corrosion current. This observation is particularly relevant for the System 4 slabs with galvanized bars in both mats. Little macrocell corrosion current was measured, yet the top-mat galvanized bars were extensively covered with zinc corrosion products. Measured macrocell currents obviously do not account for all observed corrosion phenomena. However, the data in Fig. 8 show a reasonable correlation between cumulative amp-hours of macrocell corrosion current and percent of area corroded for Systems 1, 6, 7, 7A, and 9.

Chloride Profiles After 44 Weeks of Cyclic Testing

General - The chloride ion contents from the 119 specimens that were sampled after 44 weeks of cyclic testing are tabulated in Table 8 in Appendix C and summarized in Table 5. Table 5 shows the number of slabs tested per condition, the mean chloride ion content, and the standard deviation and the coefficient of variation of the measured chloride ion contents for each condition, when at least four slabs were tested.

Effect of cover and w/c ratio on unprotected concrete - The average chloride ion contents after 44 weeks of testing for the 90 slabs that utilized unprotected concrete (Systems 1 through 5, 7 and 8) are shown in Fig. 9 plotted versus the clear cover depth. These data show very large differences in chloride ion contents at the 1-in. depth for the three w/c ratios. At the 2- and 3-in. depths, the differences in chloride ion due to variation in w/c ratio are quite low. The chloride ion contents at the 2- and 3-in. depths are less than 0.02 percent, irrespective of the w/c ratio. The chloride ion content has not reached the corrosion threshold values at the 2- or 3-in. depths at any w/c ratio. These data explain why the slabs with 2- or 3-in. cover showed no detectable evidence of corrosion activity during these cyclic tests.

These chloride ion data and the FHWA time-to-corrosion study(12) both show that significant decreases in chloride ion contents occur during exposure when the nominal w/c ratio is lowered from 0.50 to 0.40. These data support the current AASHTO requirements for a maximum w/c ratio of 0.44 for chloride environments. Fig. 10 shows a comparison of the net absorbed chloride ion contents for the nominal 0.60, 0.50 and 0.40 w/c ratio concretes used in a previous FHWA study(12) compared with the present data for the nominal 0.50, 0.40 and 0.32 w/c ratio concretes after 44 weeks of cyclic testing. These data show the same large differences in net absorbed chloride ion contents, particularly at 1 in. when comparing

Table 5 - Summary of average chloride content data at 44 weeks.

Actual W/C	Cover, in.	No. of slabs	Mean value, % by wt. of concrete	Standard deviation, % by wt. of concrete	Coefficient of variation, %
NORMAL CONCRETE - UNPROTECTED - SYSTEMS 1, 2, 3, 4, 5, 7 AND 8					
0.51	1	16	0.451	0.061	13.4
	2	16	0.0193	0.0135	69.9
	3	14	0.0106	0.0074	69.8
0.40	1	4	0.0973	0.050	51.6
	2	14	0.0064	0.0036	56.3
	3	4	0.0043	0.0005	11.6
0.28	1	4	0.025	0.0096	38.4
	2	14	0.0113	0.0077	68.1
	3	4	0.0065	0.0019	29.2
CALCIUM NITRITE CONCRETE - SYSTEM 6					
0.53	1	1	0.232	—	—
	2	2	0.008	—	—
	3	2	0.006	—	—
0.44	2	2	0.010	—	—
0.34	2	2	0.006	—	—
SILANE TREATED CONCRETE - SYSTEM 9					
0.51	1	2	0.021	—	—
	2	2	0.0085	—	—
	3	2	0.008	—	—
0.40	2	2	0.012	—	—
0.28	2	2	0.007	—	—
METHYL METHACRYLATE COATED CONCRETE - SYSTEM 10					
0.51	1	2	0.445	—	—
	2	2	0.0155	—	—
	3	2	0.0165	—	—
0.40	2	2	0.0105	—	—
0.28	2	2	0.007	—	—

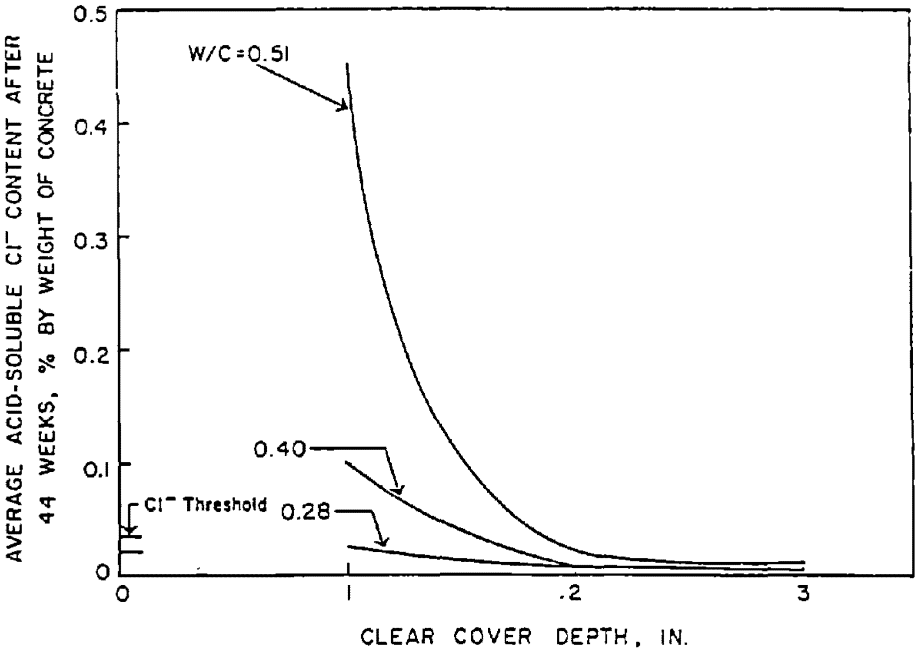


Fig 9 - Chloride content profiles at 44 weeks for different w/c ratio concretes.

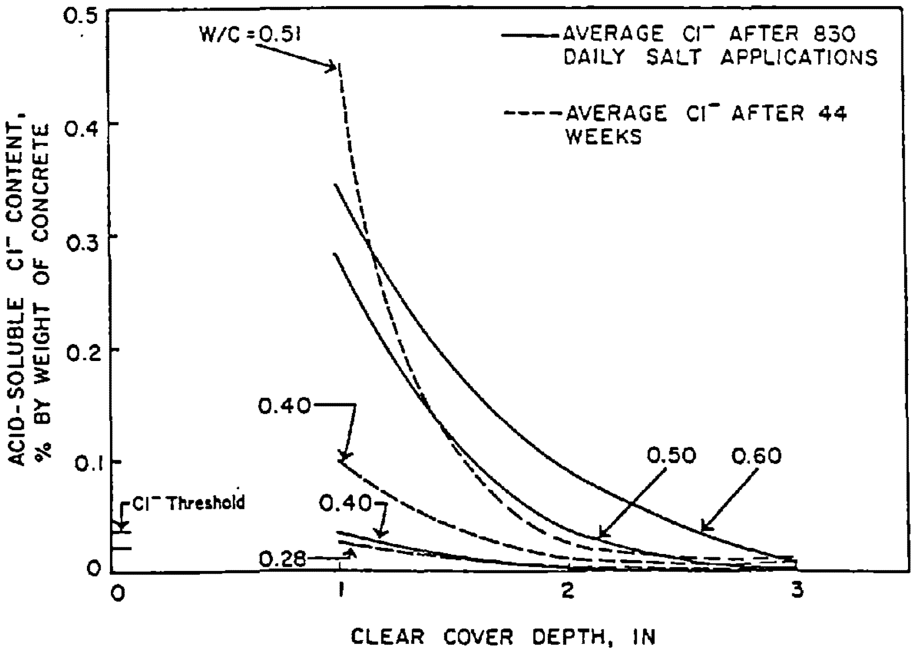


Fig. 10 - Chloride content profiles from 1976 FHWA study and present study.

the 0.50 and 0.40 concretes. These data also show that, after the 44-week cyclic test, about 60 percent more chloride ion was present at the 1-in. level with 0.50 w/c ratio concrete and 200 percent more chloride ion at the 1-in. level with the 0.40 w/c ratio concrete, compared to the 830 applications of 3 percent sodium chloride used in the previous FHWA study.

Calcium nitrite concrete - The chloride content at the 1-in. depth with the nominal 0.50 w/c ratio concrete was 44 percent less than the comparable average value from conventional concrete.

Silane-treated concrete - The average chloride ion contents at all locations are less than 0.021 percent. This is below the corrosion threshold value. The data show that the silane treatment stopped 95 percent of the chloride ion ingress at the 1-in. depth level with the 0.50 w/c ratio when compared to unprotected specimens. This reduction is essentially equal to the 97 percent reduction reported in the NCHRP 244 report.(17)

Methyl methacrylate-coated concrete - The data show that after 44 weeks of cyclic testing, the chloride ion contents are comparable to unprotected concrete at the 1-in. depth. This observation does not conform to the results in NCHRP 244,(17) where this same coating stopped 99 percent of the chloride ion with a similar concrete and depth of sampling. An examination of the coated surfaces after 48 weeks of cyclic testing showed evidence of disintegration of the coating.

CORROSION TESTS ON FULL-SIZE REINFORCED AND PRESTRESSED CONCRETE BRIDGE MEMBERS

Introduction

The purpose of the full-size member tests was to evaluate corrosion protection systems on relatively large reinforced concrete columns and beams; precast, pretensioned, prestressed concrete piles; and stay-in-place, precast, prestressed bridge deck panels. The systems evaluated were the most promising corrosion protection systems identified in the Pilot Time-to-Corrosion Study. These different bridge members are subjected in the field to chloride-laden water and snow from various sources, as listed below.

- Saltwater splash and spray in marine environments.
- Daily tidal variations on piling in seawater.
- Snowplows which throw chloride-laden snow and water against concrete bridge substructure members.
- Deicing salts applied on the bridge deck.
- Drains on bridge decks which allow saltwater from deicing salt to flow down the sides of beams or girders.
- Leaking expansion joints which allow saltwater from deicing salts to flow down onto the tops of piers and beams and then down the sides of piers, beams, columns and walls.

Deterioration of these various bridge members from the above exposure conditions is evident in many regions of the United States.

Because most laboratory corrosion studies in the past, including the Pilot Time-to-Corrosion Study, have used horizontal slab-type specimens with ponded saltwater on the top surface, these full-size studies departed from this normal ponding procedure to determine if the type of saltwater application procedure had any influence on corrosion behavior on full-size members. The saltwater was applied to these various full-size members on horizontal and vertical surfaces by causing the saltwater to flow over them. This flowing saltwater application was believed to realistically simulate more closely actual field exposure conditions. Four types of concrete members were tested, as described below:

1. Conventionally reinforced 18-in. square by 6-ft long columns which were exposed to cyclic saltwater flow down one vertical face to simulate the "snow plow" effect.
2. Conventionally reinforced 18-in. square by 4-ft long beams which were exposed to cyclic saltwater flow across the top horizontal surface and then down one vertical face. This exposure simulated the "leaking expansion joint or bridge deck drain" effect.
3. Precast, prestressed 16-in. square by 8-ft long concrete piles which were exposed to constant saltwater ponding, cyclic saltwater flow, and no saltwater exposure on one horizontal surface. This test simulated the tidal effect, which produces a constantly wet region, an alternating wet and dry region and a constantly dry region along the length of a pile.
4. Precast, prestressed 2 1/2-in. thick by 8-ft long stay-in-place bridge deck panels, with a composite 5 1/2-in. thick cast-in-place reinforced concrete wearing layer, were subjected to cyclic saltwater flow over their bottom surfaces. This exposure simulated the bottom surface saltwater splash and spray effect which is common to coastal bridge decks.

A total of 19 full-size specimens were tested during approximately 370 daily cycles of saltwater exposure and air drying in the laboratory. These specimens generally were made with 1-in. clear cover so that corrosion could possibly be initiated in the control specimens within a reasonable time period. The Pilot Study showed that 2-in. clear cover would probably preclude corrosion in any of the systems within a one-year period. Two full-size specimens, one each with 1 1/2- and 2-in. cover over the gray reinforcing bars, were used to verify this assumption.

The concrete w/c ratio was shown in the Pilot Study to have a significant influence on chloride penetration and subsequent severity of corrosion. Therefore, essentially all specimens were made with the

0.44 w/c ratio as currently specified by AASHTO. One specimen, made with a composite silica fume-based admixture, had a water to cement plus silica fume ratio of 0.18.

The reinforced concrete columns and beams were cured using moist curing methods. The precast, prestressed concrete piles and bridge deck panels were cured overnight at 130 to 140°F, using standard accelerated heat curing methods. This variation in curing technique was incorporated into these tests to determine the effect of the curing method on corrosion behavior and chloride penetration.

The corrosion protection systems that were selected for these full-size member tests, based upon the Pilot Study conclusions, are identified in Table 6. They include the following parameters:

- Epoxy-coated bars with 1-in. cover.
- Epoxy-coated strands with 1-in. cover.
- Galvanized bars with 1-in. cover.
- Silane-treated concrete with 1-in. cover over gray bars and strands.
- Calcium nitrite admixture with 1-in. cover over gray bars and strands.
- Silica fume concrete with 1-in. cover over gray bars.
- Gray bars with 1-, 1 1/2- and 2-in. cover.
- Gray strands with 1-in. cover.

As shown in Table 6, six columns, five beams, four piles and four bridge deck specimens were tested using various selected corrosion protection systems.

Table 6 - Details of full-size specimen test program.

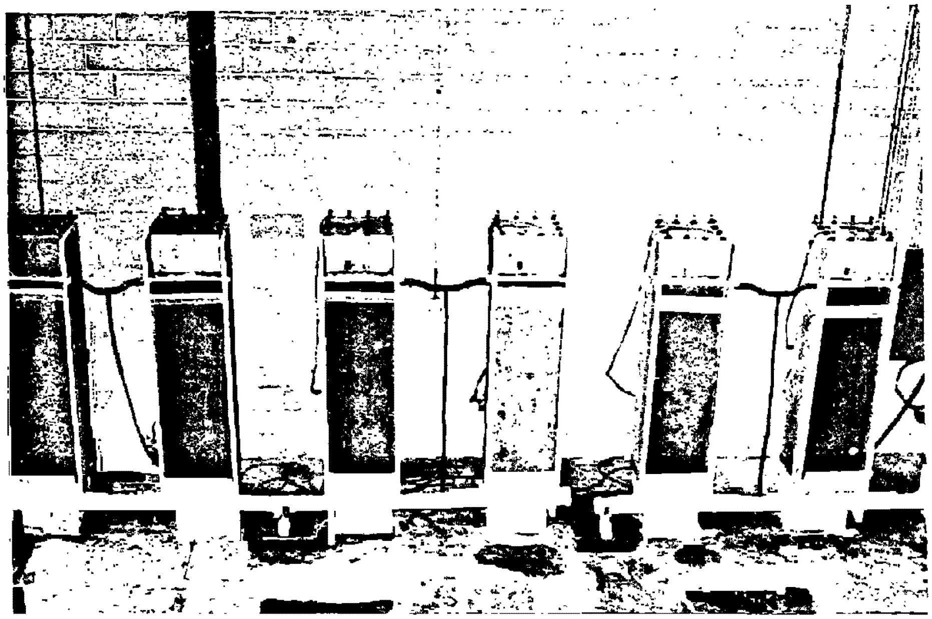
Specimen Number	Clear cover (in.)	Type of bar or strand	Description and other parameters
REINFORCED CONCRETE COLUMNS			
C 1	1	Gray	Control specimen
C 2	1	Galvanized	Galvanize all reinforcement
C 3	1	Gray	Silane treatment of concrete
C 4	1	Epoxy coated	
C 5	1 1/2	Gray	Increase clear cover to 1 1/2 in.
C 6	1	Gray	Concrete contains calcium nitrite admixture
REINFORCED CONCRETE BEAMS			
B 1	1	Gray	Control specimen
B 2	1	Gray	Low w/c ratio concrete cast with silica fume
B 3	2	Gray	Increase clear cover to 2 in.
B 4	1	Epoxy coated	
B 5	1	Gray	Silane treatment of concrete
PRECAST, PRESTRESSED CONCRETE PILES			
P 1	1	Bare	Control specimen
P 2	1	Bare	Short length corrosion current pickup probes in various zones
P 3	1	Bare	Concrete contains calcium nitrite admixture
P 4	1	Epoxy coated	
PRECAST, PRESTRESSED CONCRETE STAY-IN-PLACE BRIDGE DECK PANELS			
BD 1	1	Bare	Control specimen. Use electrical connection to simulate shear connectors
BD 2	1	Bare	Concrete contains calcium nitrite admixture. Use electrical connection to simulate shear connectors
BD 3	1	Bare	Use silane treatment on bottom of panel. Use electrical connection to simulate shear connectors
BD 4	1	Bare	Eliminate electrical connection to simulate lack of shear connectors

Specific Test Details

Snowplow effect tests on reinforced concrete columns - As shown in Fig. 11, the conventionally reinforced concrete columns were 6 ft long and 18 in. square. Each specimen contained twelve No. 6 reinforcing bars having 1- or 1 1/2-in. cover, as shown in Table 6. The 4 hr per day saltwater flow was restricted to one vertical face while the other three vertical faces were always exposed to air. The saltwater flow path was restricted to the lower 5-ft portion of the 6-ft tall column. The four potentially anodic bars, shown in Fig. 11, were on the exposed test surface while the eight potentially cathodic bars were located on the surfaces exposed to air.

Ponding and gravity saltwater flow tests on reinforced concrete beams - As shown in Fig. 12, the conventionally reinforced concrete beams were 4 ft long and 18 in. square. Each specimen contained twelve No. 6 reinforcing bars having 1- or 2-in. cover, as shown in Table 6. The 4-hr per day saltwater flow was restricted to the nearly horizontal top surface and one vertical surface. The other two surfaces were always exposed to air. The test incorporated seven potentially anodic bars and five potentially cathodic bars, as illustrated in Fig. 12.

Tidal variation tests on precast, prestressed piles - As shown in Fig. 13, the prestressed piles were 8 ft long. A 1-ft length at each end was 18 in. square in cross-section while the center 6-ft long test section was 16 in. square. The end-of-pile thickening was used to minimize longitudinal cracking during detensioning since no circumferential spirals or ties were used. The piles were tested in the horizontal position. The "dumbbell shape" is indicated in Fig. 13, which illustrates the dimensions and details of the saltwater exposure tests. Eight lengths of fully stressed 1/2-in. strands with 1-in. cover were used in each pile. Each pile contained three potentially anodic and five potentially cathodic prestressing strands. Two full-length No. 4 gray bars having 1-in. cover



REINFORCED CONCRETE COLUMNS

- 18 x 18 x 72 inch column
- 1.0 and 1.5 inch cover over No.6 bars

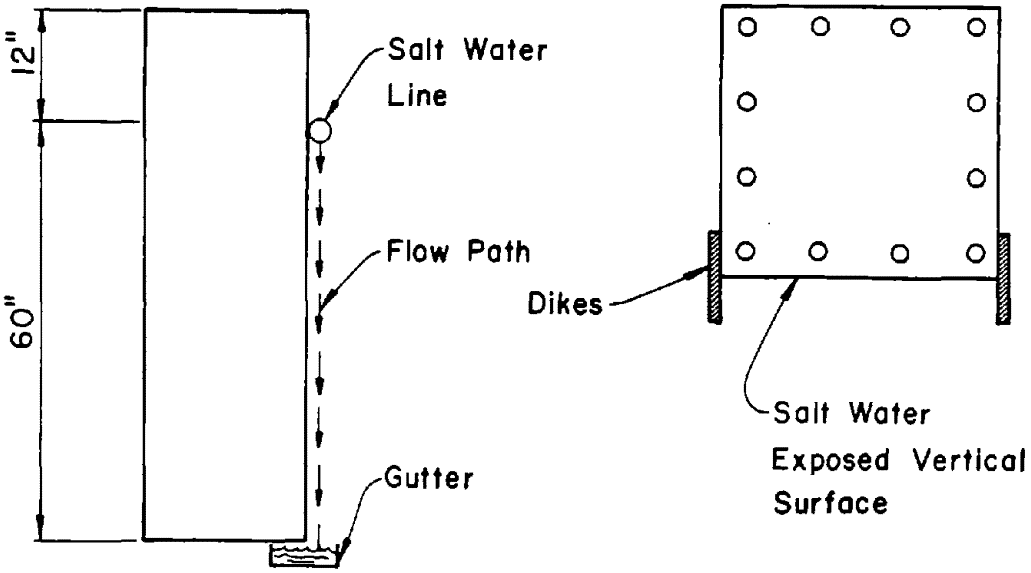
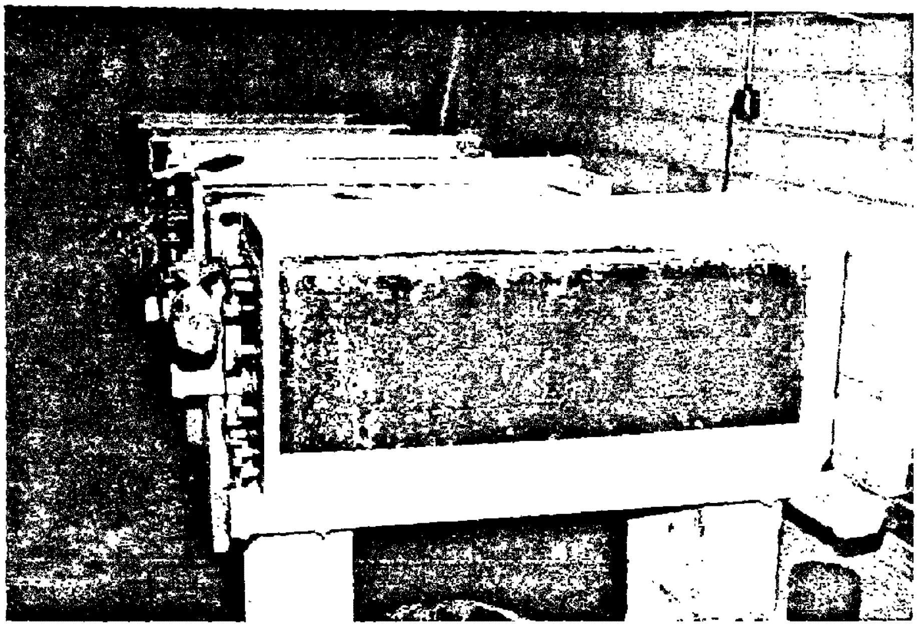


Fig. 11 - Details of cyclic tests on columns.



REINFORCED CONCRETE BEAMS

- 18 x 18 x 48 inch beams
- 1.0 and 2.0 inch clear cover over No. 6 bars

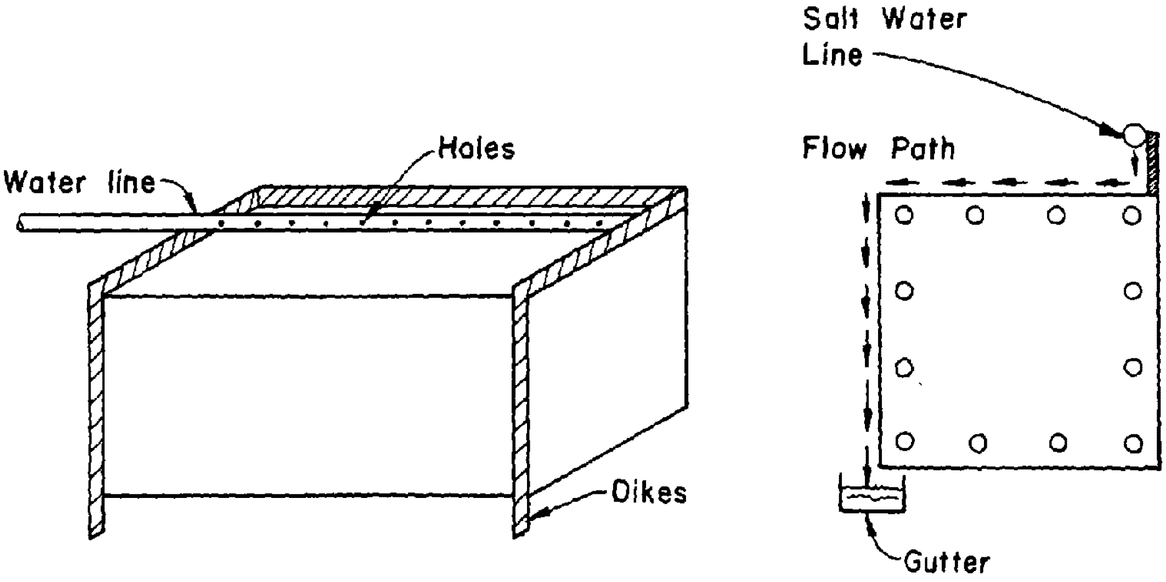
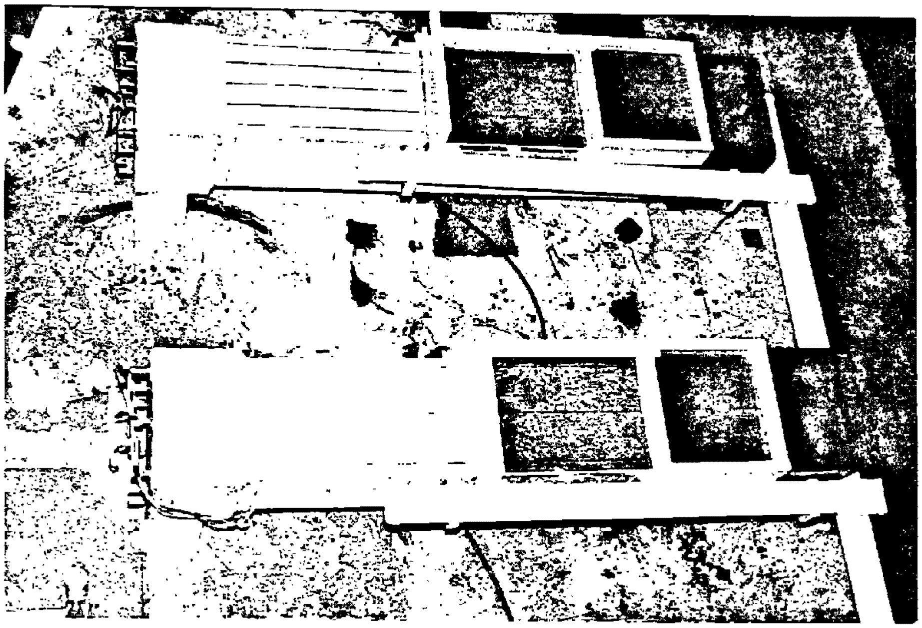


Fig. 12 - Details of cyclic tests on beams.



PRECAST, PRESTRESSED CONCRETE PILES

- 16 x 16 x 96 inch piles
- 1.0 inch cover over 1/2 inch strand (O) and No. 4 bars (■)

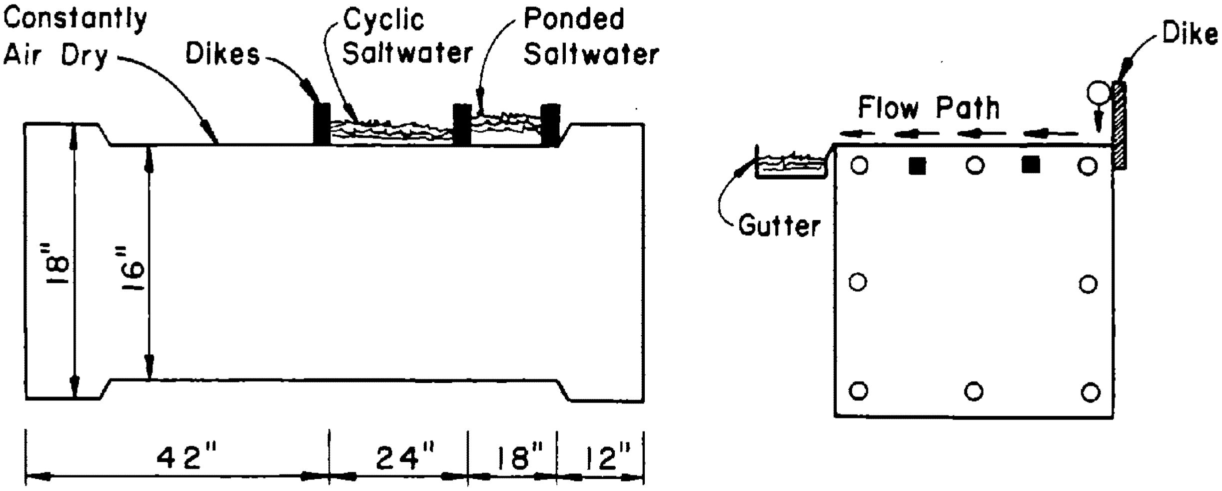
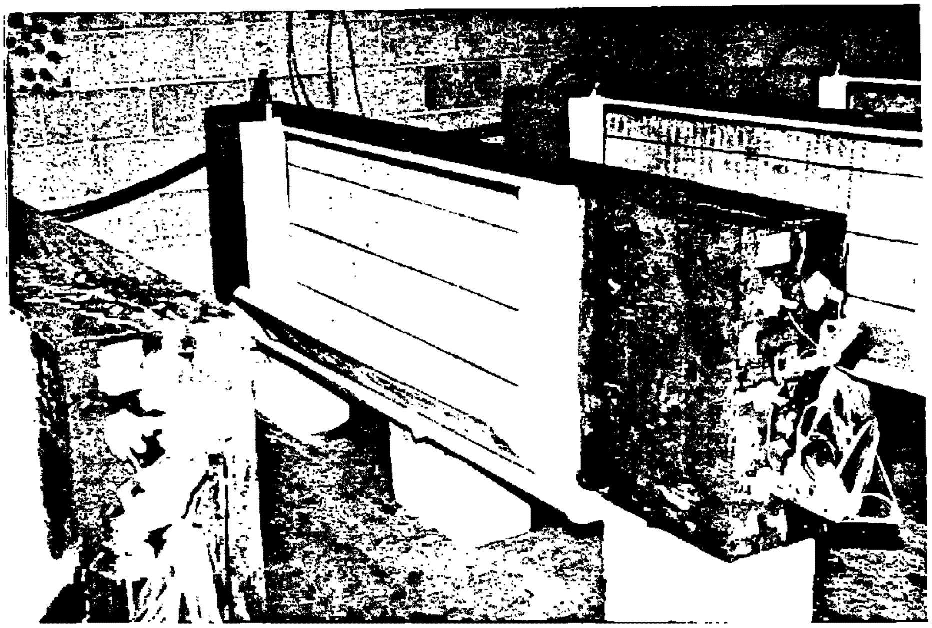


Fig. 13 - Details of cyclic tests on piles.

were also located in the test area between the three anodic strands. These supplemental bars were included to determine if the time-to-corrosion or degree of corrosion of mild steel was different from that of highly-stressed prestressing steel. These full-length No. 4 bars were contained in Specimens P1, P3 and P4. Specimen P2 contained eight short-length (5 1/2 in.) embedded reinforcing bar corrosion probes. These were also located between the three anodic strands, in pairs, in the ponded area, crossing the junction of the constantly ponded area and cyclic wet/dry area, in the wet/dry area, and in the constantly dry area. Each of these eight probes was connected with lead wires to the external buss bar connections on the other reinforcing with similar exposure in the pile. Specimen P2 also contained three specially manufactured fusion bonded epoxy-coated strands. A noncoated 37-in. length of each was positioned across the 24-in. cyclic wet/dry test region. The top three strands in P2 were these specially manufactured strands. The lower five strands were normal, totally bare strands.

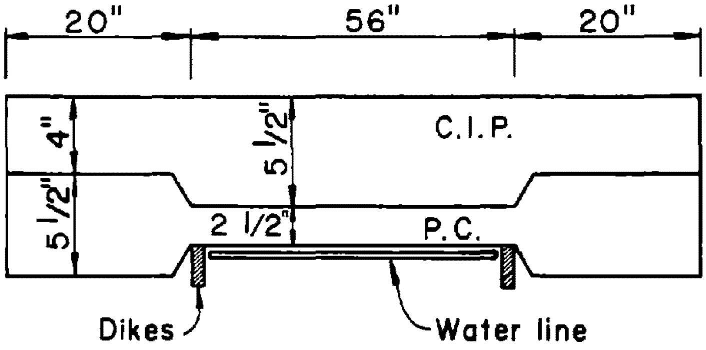
The constantly ponded saltwater test area was 18 in. long. This region represented the submerged portion of a pile. The cyclic saltwater test area was 24 in. long. This area was subjected to 6 hrs of flowing saltwater, followed by 6 hrs of air drying on a continual 24-hr basis. The constantly dry region was 42 in. long.

Saltwater spray tests on precast, prestressed subdeck panels - As shown in Fig. 14, the 8-ft long precast, prestressed concrete stay-in-place bridge deck panel and composite cast-in-place concrete wearing layer were positioned with their widths in the vertical position. This test position was considered to be more severe than the upside-down horizontal position. Capillary pressures pulling the saltwater into the concrete are much greater than the pressures due to gravitational forces. Therefore, it was concluded that the position of the exposed surface probably did not make a significant difference in the ability of the saltwater to penetrate into the concrete.



PREÇAST, PRESTRESSED CONCRETE STAY-IN-PLACE BRIDGE DECK PANELS

- 2 1/2 x 24x96 inch deck panel
- 5 1/2 x24x 96 inch C.I.P. concrete layer
- 1.0 inch cover over 1/2 inch strand (O) and No. 4 bar (●)



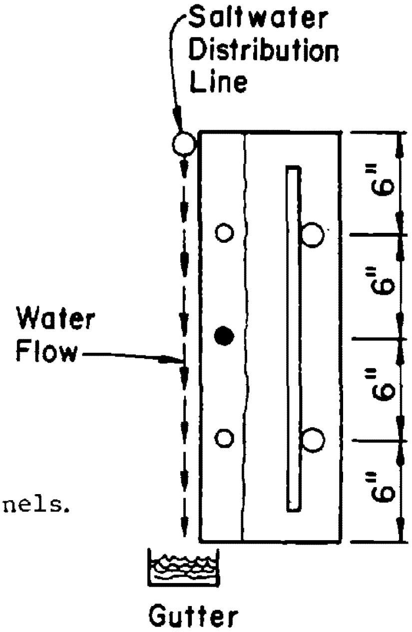


Fig. 14 - Details of cyclic tests on bridge deck panels.

The dumbbell-shaped axially prestressed, precast subdeck panels were about 56 in. long, 24 in. wide and 2 1/2 in. thick in the cyclic test region. Each panel contained two fully-stressed 1/2-in. prestressing strands with 1-in. cover as shown in Table 6. Each panel also contained one full-length No. 4 gray reinforcing bar positioned between the two prestressing strands, also having 1-in. cover. The 5 1/2-in. thick cast-in-place layer contained a potentially cathodic normal reinforcing bar mat containing No. 6 longitudinal and lateral reinforcing bars. None of these specimens contained steel shear connectors between the two layers of concrete. The 4 hr per day saltwater flow was restricted to the vertical surface of the precast concrete subdeck panel, as shown in Fig. 14.

Saltwater exposure - The salt solution was 15 percent sodium chloride in water. This solution contains about 9.1 percent chloride ion by weight, a value about 5 times that of normal seawater. This recirculating saltwater solution was routinely monitored, adjusted or changed to maintain the proper chloride level. All tests were performed at temperatures of 60 to 80°F. During the winter season when the relative humidity of the air in the heated laboratory was low, these full-size specimens were thoroughly washed with fresh water every two weeks. This water washing was undertaken to maintain the concrete surfaces exposed to air at high relative humidities.

The saltwater was circulated over the selected test surfaces on the columns, beams and bridge deck specimens for 4 hrs each day for approximately 370 days. These specimens were allowed to air dry for the remaining 20 hrs each day.

The 370-day saltwater exposure testing of the prestressed piles consisted of a 6-hr saltwater circulating period alternating with a 6-hr air drying period on a continuous 24-hr basis for the simulated tidal region. The simulated submerged region was constantly ponded, and the simulated above-tidal zone region was constantly in air.

The saltwater was circulated over each specimen at a rate averaging about 1 1/2 gals per min.

Electrical corrosion measuring systems - The electrical measurements made on these simulated full-size bridge members were accomplished in the same way as those measured on the small slabs in the Pilot Study. The ends of the reinforcing steel were exposed at one end of each specimen and electrical continuity between all reinforcement was made through an external network of lead wires and resistors. Switching permitted selective interconnection of anodic and cathodic reinforcement, if desirable. With this system, it was possible to measure macrocell corrosion currents, instant-off potentials, and electrical resistances between individual lengths of reinforcement or between groups of anodic and cathodic reinforcement in these large specimens. Copper-copper sulfate half-cell potentials were also measured on the concrete surfaces. These various corrosion activity measurements generally were made on a monthly basis.

Steel shear connectors are commonly used between stay-in-place bridge deck panels and the cast-in-place composite wearing layer. This steel provides a direct electrical path between potentially anodic prestressing strands and potentially cathodic reinforcing bars in the cast-in-place wearing surface layer. This direct electrical path was eliminated in Specimen BD4 to determine how the corrosion process was influenced by the lack of the typical macrocell electrical connection between a large cathodic region and a potentially anodic region. Actual shear connectors were not used in any of these four precast bridge deck specimens. The other three bridge deck specimens were provided with a full-time external lead wire connection between the reinforcing in both sections, which would simulate the effect of the steel shear connectors.

General details - The details on the reinforcing steels, concrete mixtures, specimen fabrication, chloride ion tests, and coating procedures are given in Appendix D.

Test Results and Discussion

The test results will be discussed in the following sections. They will deal with overall corrosion behavior of the reinforced concrete columns and beams, overall corrosion behavior of the precast, prestressed concrete piles and bridge deck panels, AC resistance of the different specimen types, chloride content profiles after the cyclic testing, and half-cell and corrosion current relationships for full-size specimens.

Overall Corrosion Behavior of Reinforced Concrete Columns and Beams

The four specimens which showed measurable corrosion activity from the corrosion measurement systems were as follows:

- Gray bars with 1-in. cover (C1 and B1).
- Galvanized bars with 1-in. cover (C2).
- Calcium nitrite concrete with 1-in. cover over gray bars (C6).

The seven specimens which showed no measurable corrosion activity were as follows:

- Gray bars with 1 1/2- and 2-in. cover (C5 and B3).
- Epoxy-coated bars with 1-in. cover (C4 and B4).
- Silane-treated concrete with gray bars with 1-in. cover (C3 and B5).
- Silica fume concrete with gray bars with 1-in. cover (B2).

A description of the significant data and conclusions obtained from each of these eleven conventionally reinforced concrete columns and beams follows. Those which exhibited no measurable corrosion activity are discussed first followed by discussions of the specimens which did exhibit corrosion activity.

Silane-treated concrete column - The silane-treated concrete column C3 with 1-in. cover over gray bar exhibited no corrosion activity. The average final chloride ion contents at the 1/2- and 1-in. depths were 0.027 and less than 0.004 percent, respectively. Corrosion would not be anticipated because the final chloride content at the depth of the bars (0.004 percent) was less than the corrosion threshold level. The bars were examined at the conclusion of the testing and were found to be free of corrosion.

The 0.027 percent chloride content at the 1/2-in. depth of the silane-treated specimen is 5 percent of the average chloride content at that depth in specimens C1, C2, C4 and C5. The latter four contained the same concrete, but were not silane-treated. The chloride content at the 1-in. depth of column C3, less than 0.004 percent, is about 1 percent of the average chloride content of specimens C1, C2, C4 and C5. These data show that the silane treatment stopped essentially all ingress of chloride-laden water to the 1 in. or more depths. It also impeded 95 percent of the ingress to the 1/2-in. depth, compared to unprotected concrete.

Column with epoxy-coated bars - The fusion bonded epoxy-coated bars with 1-in. cover in C4 exhibited no corrosion activity. The average final chloride ion contents at the 1/2- and 1-in. depths were 0.522 and 0.240 percent, respectively. At the 1-in. level, the final chloride content of 0.240 percent was over 8 times the chloride corrosion threshold level. The bars were examined after testing and were found to be free of corrosion. Even holiday locations, premarked on the bars at the time of casting, were found to be free of any corrosion products.

Column with gray bars with 1 1/2-in. cover - Column C5, which had 1 1/2-in. of clear cover over gray bars, exhibited no corrosion activity. The average final chloride ion contents at the 1/2-, 1- and 1 3/4-in. depths were 0.678, 0.330 and 0.043 percent, respectively. Special chloride content tests on additional concrete samples were also made at the 1 3/8- and 1 1/2-in. depths to better define the chloride contents at the bar surface. The average chloride contents at these two depths were 0.13 and 0.02 percent, respectively. After reviewing the chloride content data at the 1 3/4-in. depth from all the columns, beams, piles and bridge decks, as well as the data from the two special tests, the original two C5 chloride content data points of 0.074 and 0.076 percent at the 1 3/4-in. depth were omitted as being questionable. The average chloride content at the 1 1/2-in. depth was determined from a graphical plot, using seven data points, to be about 0.02 percent. This is less than the chloride corrosion threshold level of 0.030 percent. The bars were examined and were found to be free of corrosion by-products.

Silica fume concrete beam - Beam B2, with 1-in. cover over gray bars, exhibited no corrosion activity. The average final chloride ion contents at the 1/2- and 1-in. depths were 0.100 and 0.006 percent, respectively. Corrosion would not have been anticipated because the chloride content at the level of the bars (0.006) was less than the corrosion threshold level of approximately 0.04 percent for this particular concrete mixture. The bars were examined and were found to be free of corrosion.

The chloride content at the 1/2- and 1-in. depths of 0.100 and 0.006 are 19 and 2 percent, respectively, of the average chloride contents at the same depths in specimens B1, B3 and B4, which utilized the conventional 0.44 w/c ratio concrete. These data show that the silica fume concrete mixture stopped essentially all ingress of chloride-laden water to the 1 in. or more depth levels. It also stopped about 80 percent of the salt water ingress to the 1/2-in. depth, compared to the conventional concrete.

Beam with gray bars with 2-in. cover - Beam B3, with 2 in. of clear cover over gray bars, exhibited no corrosion activity. The average final chloride ion contents at the 1/2-, 1-, 1 3/4- and 2 1/2-in. depths were 0.543, 0.279, 0.012 and 0.005 percent, respectively. The average chloride content at the 2-in. depth was determined from a graphical plot to be 0.010 percent. This value is less than the chloride corrosion threshold level of about 0.03 percent, and corrosion, therefore, would not have been anticipated. The bars were examined and were found to be free of corrosion.

Beam with epoxy-coated bars - The fusion bonded epoxy-coated bars with 1-in. cover in specimen B4 exhibited no corrosion activity. The average final chloride ion contents at the 1/2- and 1-in. depths were 0.541 and 0.268 percent, respectively. At the 1-in. level, the final chloride content of 0.268 percent was over 9 times the chloride corrosion threshold level. The bars were examined and found to be free of corrosion.

Silane-treated concrete beam - The silane-treated concrete beam B5, with 1-in. cover over gray bars, exhibited no signs of active corrosion. The average final chloride contents at the 1/2- and 1-in. depths were both less than 0.004 percent. These data show that the silane treatment stopped essentially all ingress of chloride-laden water to the 1/2 in. or more depths. Corrosion would not be anticipated under these conditions and the examined bars were free of corrosion.

Column with gray bars with 1-in. cover - The conventionally reinforced concrete column C1 with 1-in. cover over gray bars showed evidence of significant active corrosion from the electrical measuring systems. It started after less than 25 daily cycles or in less than 7 percent of the total test period. Half-cell potentials after about 30 and 370 daily cycles ranged from -0.32 to -0.48 volts and from -0.41 to -0.52 volts, respectively. Corrosion current and instant-off potential data as a function of time for the bar showing the greatest corrosion activity are shown in Fig. 15.

The average final overall chloride contents at the 1/2- and 1-in. depths were 0.553 and 0.307 percent, respectively. The average final

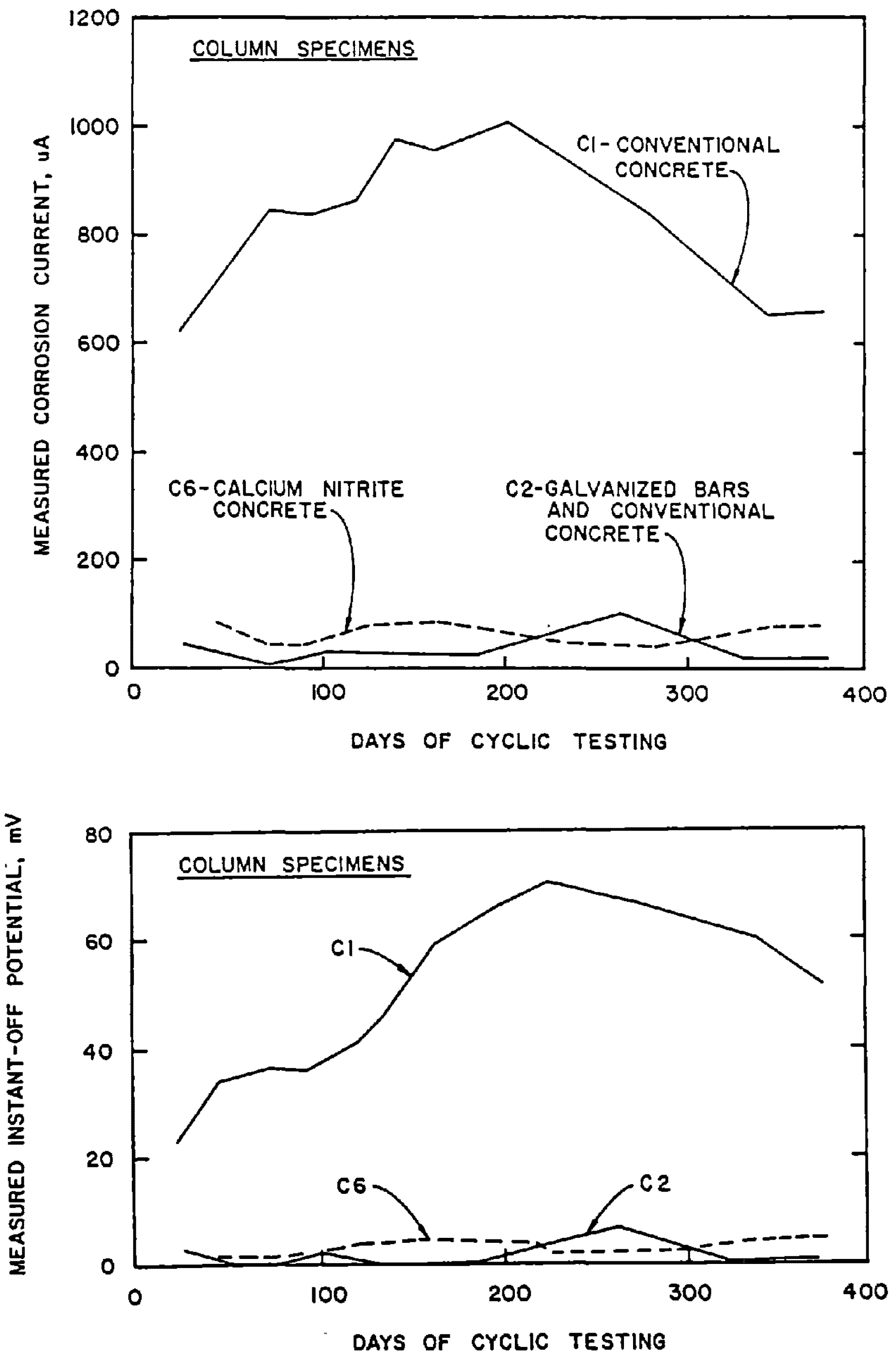


Fig. 15 - Corrosion activity data from reinforced concrete columns.

chloride content at the depth of the reinforcing bars was about 10 times the chloride corrosion threshold. The four potentially anodic bars were examined. The area of corrosion by-products on the bar half-perimeter closest to the saltwater-exposed surface ranged from 2 to 22 percent. The average was 11 percent of that half-perimeter area. This 11 percent surface area is much less than the 73 percent surface area measured on the 0.50 w/c ratio specimens with 1-in. cover from the Pilot Study. However, the 11 percent value is essentially the same as the average 9 percent surface area measured on the 0.40 w/c ratio specimens with 1-in. cover from the Pilot Study. This similarity tends to confirm the beneficial effect of lower w/c ratio concretes as also shown in the Pilot Study.

Column with galvanized bars - The reinforced concrete column C2, with 1-in. cover, showed evidence of zinc corrosion in less than 25 daily cycles. The subsequent corrosion current was low, however. Half-cell potentials after about 30 and 370 daily cycles were about -0.68 and -0.58 volts, respectively. Corrosion current and instant-off potential data for the bar showing the greatest corrosion activity are shown in Fig. 15.

The average final overall chloride contents at the 1/2- and 1-in. depths were 0.580 and 0.322 percent, respectively. Thus, the average final chloride content at the level of the galvanized bars was about 10 times the corrosion threshold level for gray bars. The four potentially anodic galvanized bars were examined. The surface of these four bars were covered with whitish-colored and blackish-colored zinc oxidation products. Small, nominal 1/8-in. diameter red iron oxide rust stains were located at four spots where the zinc had corroded away and minor steel corrosion had started.

Column with calcium nitrite admixture concrete - The reinforced calcium nitrite concrete column C6, with 1-in. cover, showed evidence of minor corrosion activity from the measuring systems in less than 25 daily cycles. This is less than 7 percent of the total test period. Half-cell potentials after about 30 and 370 daily cycles ranged from -0.24 to -0.27 volts and from -0.26 to -0.29 volts, respectively. Corrosion currents and instant-off potentials as a function of time for the bar showing the greatest corrosion activity are shown in Fig. 15.

The average final overall chloride contents at the 1/2- and 1-in. depths were 0.608 and 0.319 percent, respectively. While the final chloride content at the level of the bars was about 10 times the corrosion threshold level, the measured corrosion current data indicated very minor corrosion activity. This is shown in Fig. 15. The four potentially anodic bars were examined. The amount of corrosion by-product on the bar half-perimeter ranged from 0 to 4 percent and averaged only 1.25 percent. The data show that the amount of corrosion by-products on the steel in the 0.44 w/c ratio calcium nitrite concrete column C6 was about one-tenth that measured on the bars from the conventional concrete column C1. This same one-tenth factor was observed in the Pilot Study slabs which incorporated 0.50 w/c ratio concrete and 1-in. cover, i.e., 73 percent versus 7.5 percent corroded areas. This similarity of data substantiates the beneficial effect of using the calcium nitrite admixture as was also concluded in the Pilot Study. These cumulative data also show the beneficial effect of using a lower w/c ratio with calcium nitrite mixtures.

Beam with gray bars with 1-in. cover - The reinforced concrete beam B1 showed evidence of corrosion activity in less than 25 days. Half-cell potentials after about 30 and 370 days were about -0.21 and -0.43 volts, respectively. Corrosion currents and instant-off potentials for the bar showing the greatest corrosion activity are shown in Fig. 16.

The average final overall chloride contents at the 1/2- and 1-in. depths were 0.516 and 0.242 percent, respectively. The final chloride content at the 1-in. bar depth level was 8 times the corrosion threshold.

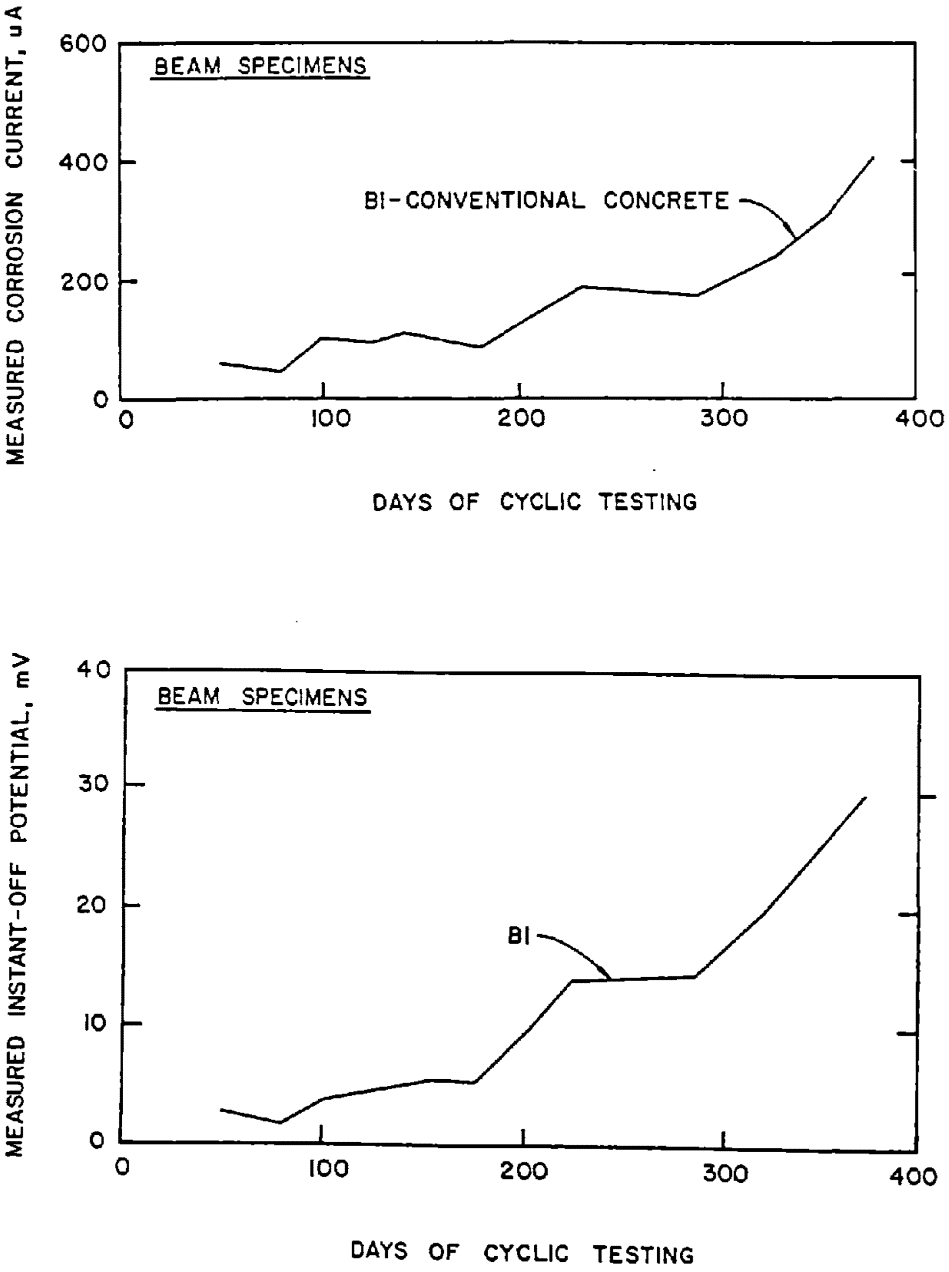


Fig. 16 - Corrosion activity data from reinforced concrete beams.

The seven potentially anodic bars were examined.The amount of corrosion by-product on the bar half-perimeter ranged from 0 to 16 percent and averaged 7 percent. This value compares with the 11 percent value measured on the bars from the conventional concrete column C1 which developed greater corrosion activity, but which had similar exposure and test conditions.

Overall Corrosion Behavior of Precast, Prestressed Concrete Piles and Bridge Deck Panels

The comparison of the overall corrosion behavior of the four prestressed concrete piles was complicated by the development during the detensioning operation at age one day of an extremely narrow, longitudinal hairline crack directly over the middle potentially anodic strand in each pile. This single crack in each pile started at both ends but was not full length of the pile. The other two potentially anodic strands in each pile as well as the two potentially anodic bars in each pile were tested in crack-free concrete. The following discussions are concerned with if and how the single longitudinal crack, which occurred prior to cyclic testing, influenced the corrosion behavior of the middle strand as compared to the corrosion behavior of the strands and bars in adjacent crack-free concrete in the piles.

The four prestressed concrete bridge deck panels did not develop any cracking during detensioning and remained crack free during the cyclic testing.

The prestressed concrete specimen types which showed measurable corrosion activity in crack-free concrete were as follows:

- Gray strands with 1-in. cover (BD1, BD4 and P2).
- Gray bars with 1-in. cover (BD4).
- Calcium nitrite concrete with 1-in. cover over gray strands (BD2 and P3).

The four specimen types which showed no measurable corrosion activity in crack-free areas were as follows:

- Silane-treated concrete with gray bars and strands with 1-in. cover (BD3).
- Epoxy-coated prestressing strands with 1-in. cover (P4).
- Gray strands with 1-in. cover (P1).

A description of the significant data and conclusions from each of these eight precast, prestressed concrete piles and bridge deck panels follows. Those specimens which exhibited no measurable significant corrosion activity are discussed first, followed by discussions of the specimens which did exhibit corrosion activity.

Pile with gray strands and bars - Pile P1, which had 1-in. clear cover over gray strands and gray bars, exhibited no measurable corrosion activity in the crack-free areas which contained two strands and two bars. The average final chloride ion contents at the 1/2- and 1-in. depths in the constantly ponded and cyclic wet/dry regions are tabulated below:

	Chloride content, %	
Depth, in.	Ponded area	Wet/dry area
1/2	0.317	0.213
1	0.197	0.081

In these two test regions, the final chloride content at the 1-in. depth was 3 to 6 times greater than the usual corrosion threshold level, yet measurable corrosion activity was not observed on the strands or bars. When the two strands were examined, the overall impression was that the strands were like new. However, when the strands were closely examined with a magnifying glass, spotty, very thin oxide points were detected. These deposits would be described as superficial, because pitting was not evident. These superficial deposits could have been present prior to fabricating the specimen or they may be the start of minute localized corrosion cells. When the two rebars were examined, one bar showed similar localized deposits, which may or may not be corrosion related.

The other bar showed minor corrosion by-products on about 4 percent of the bar half-perimeter area. This localized corrosion activity on this one bar was not detected from the macrocell measuring system. This lack of detection may relate to the large differences in corrosion current between the middle strand, beneath the crack, and the adjacent bars which both showed positive cathodic currents.

The middle strand, which was directly under the hairline crack, showed measurable corrosion activity after 50 daily cycles, as shown in Fig. 17. The crack started in the prestress transfer region. From there it extended through the constantly ponded region as well as the cyclic area. It stopped 3 in. into the constantly dry area. The eventual corrosion currents were very high. The half-cell potentials at the start and conclusion of the testing were -0.13 and -0.44 volts in the ponded area and -0.14 and -0.37 volts in the cyclic area, respectively. When the middle strand was examined, the amount of corrosion by-products on the strand half-perimeter area closest to the saltwater-exposed surface was 70 percent in the ponded area, 9 percent in the cyclic area and 0 percent in the constantly air dry area.

Pile with gray strands and gray rebar probes - Pile P2 had 1-in. clear cover over gray strands and short rebar corrosion probes. These exhibited no measurable corrosion activity in the crack-free areas until the final reading at 363 days. The average final chloride contents at the 1/2- and 1-in. depths in the constantly ponded and cyclic test areas are tabulated below:

	Chloride content, %	
Depth, in.	Ponded area	Wet/dry area
1/2	0.291	0.192
1	0.105	0.097

In these two test regions, the chloride contents were 3 times greater than the usual corrosion threshold level, yet measurable significant corrosion activity of the strands or bar was not measured. When the

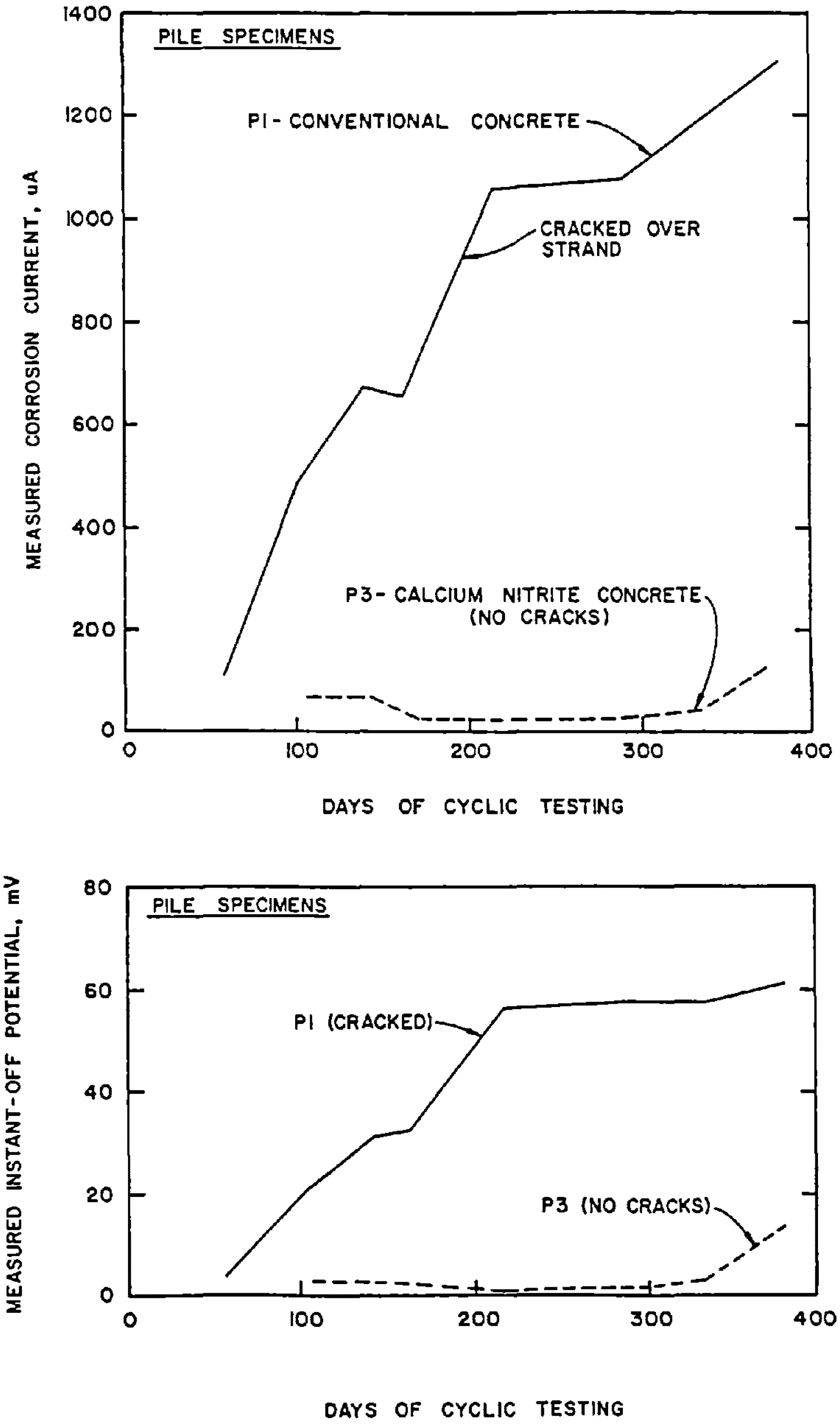


Fig. 17 - Corrosion activity data from prestressed concrete piles.

37-in. lengths of bare gray strands which crossed the wet/dry region were examined from the crack-free areas, the two strands were like new, as previously described. The 5 1/2-in. long rebar probes from the dry area, the wet/dry area, the border of the ponded and wet/dry area, and the ponded area were all examined. All eight bars were found to be essentially free of corrosion by-products.

The middle gray strand showed no measurable corrosion activity. The crack over this strand projected from one end of the pile into the ponded area for only 3 in. When the 37-in. length of the middle bare strand was examined, only very small and questionable traces of corrosion were found, in the wet/dry area.

<u>Pile with epoxy-coated strands</u> - Pile P4, which utilized 1-in. cover over epoxy-coated strands and gray bars exhibited no measurable corrosion activity in the crack-free areas. The average final chloride contents at the 1/2- and 1-in. depths in the ponded and cyclic areas are tabulated below:

Depth, in.	Chloride content, % Ponded area	Chloride content, % Wet/dry area
1/2	0.327	0.222
1	0.186	0.071

In these two test regions, the chloride contents were 2 1/2 to 6 times greater than the usual corrosion threshold level.

When the two epoxy-coated strands from the crack-free areas were examined, no corrosion by-products were found. The epoxy-coating showed no evidence of cracking or debonding from tensioning effects or from detensioning effects within the hardened concrete. The concrete had been well bonded to the sanded epoxy surface. The two gray bars were also found to be free of corrosion by-products.

The crack was almost full length of the pile and the one epoxy-coated strand beneath the crack exhibited no measurable corrosion activity. When this strand was examined, corrosion by-products were not found.

Silane-treated bridge deck - The precast concrete bridge deck panel BD3, which had silane-treated concrete with 1-in. cover over gray strands and gray bars, exhibited no measurable corrosion activity. The average final chloride contents at the 1/2- and 1-in. depths were both less than 0.004 percent. These data show that the silane treatment stopped essentially all ingress of chloride-laden water to the 1/2 in. or greater depth levels. Corrosion would not be anticipated under these conditions.

Pile with calcium nitrite concrete - The pile P3, which had 1-in. cover over gray strands and gray bars, developed corrosion activity on one strand after about 100 daily cycles in the crack-free area as shown in Fig. 17. The subsequent corrosion currents were very low, except for near the conclusion of the testing. Half-cell potentials in the ponded and wet/dry areas at the start of testing were -0.20 and -0.19 volts, respectively. They were -0.27 and -0.29 volts at the end of the testing, respectively. The average final chloride contents at the 1/2- and 1-in. depths in the ponded and cyclic areas are tabulated below:

	Chloride content, %	
Depth, in.	Ponded area	Wet/dry area
1/2	0.446	0.415
1	0.265	0.210

In these two test regions, the final chloride contents were 7 to 9 times greater than the usual corrosion threshold level at the 1-in. depth. When the two strands from crack-free concrete areas were examined, one strand showed corrosion by-products on 10 percent of the half-perimeter area in the cyclic wet/dry test region and 2 percent corrosion by-products on the same strand in the ponded region. The other strand showed only superficial localized deposits. The two bars were essentially corrosion free except for localized superficial deposits.

The crack over the middle strand extended from the one end through the ponded region and stopped as it entered the cyclic region. The middle gray strand started to show measurable corrosion activity after about 200 daily cycles. The subsequent corrosion currents were extremely small. This time-to-corrosion was longer than the 100 daily cycles required to start corrosion in the crack-free areas. The examined middle strand under this crack showed no measurable corrosion by-products in the ponded region. Only localized, superficial deposits were found in the cyclic wet/dry region.

Bridge deck with gray strands and bar - The bridge deck specimen BD1, which had 1-in. cover over gray strands and a gray bar, showed measured corrosion activity of strand after about 100 daily cycles, as shown in Fig. 18. The subsequent corrosion currents were moderately high. The gray bar did not show any evidence of corrosion activity from corrosion current measurements. The half-cell potentials were about -0.16 volts at the start of cyclic testing for both the strands and the bar. The half-cell potentials after 370 cycles for the one corroding strand was -0.44 volt. The apparently noncorroding other strand and bar regions had half-cell potentials of -0.25 and -0.22 volts, respectively.

The average final chloride contents at the 1/2- and 1-in. depths were 0.320 and 0.062 percent, respectively. The final chloride content at the 1-in. depth was 2 times greater than the usual corrosion threshold level. When the two strands were examined, one was essentially like new, except for localized, superficial deposits. The other strand had corrosion by-products on 12 percent of the half-perimeter area. The examined bar was essentially like new.

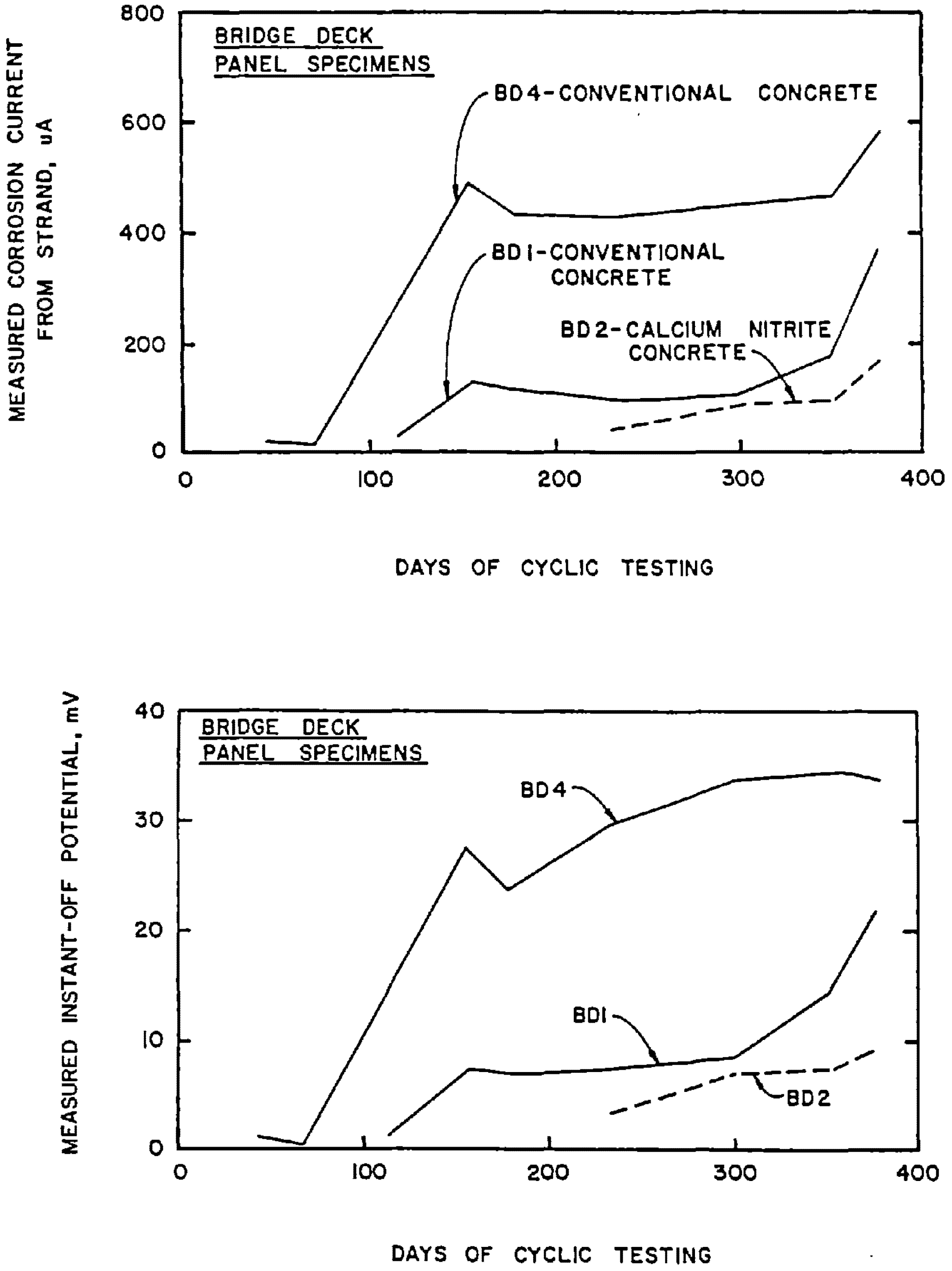


Fig. 18 - Corrosion activity data from prestressed concrete bridge deck panels.

Bridge deck with calcium nitrite concrete - The bridge deck BD2, which had 1-in. cover over gray strands and gray bar, developed corrosion activity of the two strands after about 70 and 225 daily cycles. The corrosion current and I.O.P. data are shown in Fig. 18 for the strand which developed the greatest corrosion current. The subsequent corrosion currents were relatively low. Half-cell potentials were about -0.16 volts at the start of cyclic testing for both strands and bar. The half-cell potentials after 370 cycles for the two corroding strands were about -0.26 volts.

The final average chloride contents at the 1/2- and 1-in. depths were 0.435 and 0.161 percent, respectively. The final chloride content at the 1-in. depth was about 5 times greater than the usual corrosion threshold level. When the two strands were examined, the corrosion by-products on their half-perimeter areas covered 2.7 and 3.8 percent, respectively. The bar was found to be essentially like new and noncorroded.

Conventional concrete bridge deck without macrocell connection - The bridge deck specimen BD4, which had 1-in. cover over gray strands and the gray bar, was constructed essentially the same as BD1. However, the external electrical connection between the two strands and one bar in the precast panel to the No. 6 bars in the cast-in-place concrete was not permanent. Irrespective of this lack of connection, corrosion activity of the two strands started after about 45 and 350 daily cycles. The one bar started to corrode after about 175 cycles, as shown in Fig. 19. This reinforcing bar was the only one which exhibited significant macrocell corrosion current in any of the eight precast, prestressed piles or bridge deck panel specimens. The half-cell potentials were about -0.16 volts for the strands and bar at the start of cyclic testing. As shown in Fig. 18, the subsequent corrosion currents were high for the strand, which started to corrode at 45 days. They were relatively low for the bar and the other strand. The half-cell potentials after 370 cycles for the two strands were -0.39 and -0.31 volts, respectively. The final half-cell potential of the bar was -0.29 volts.

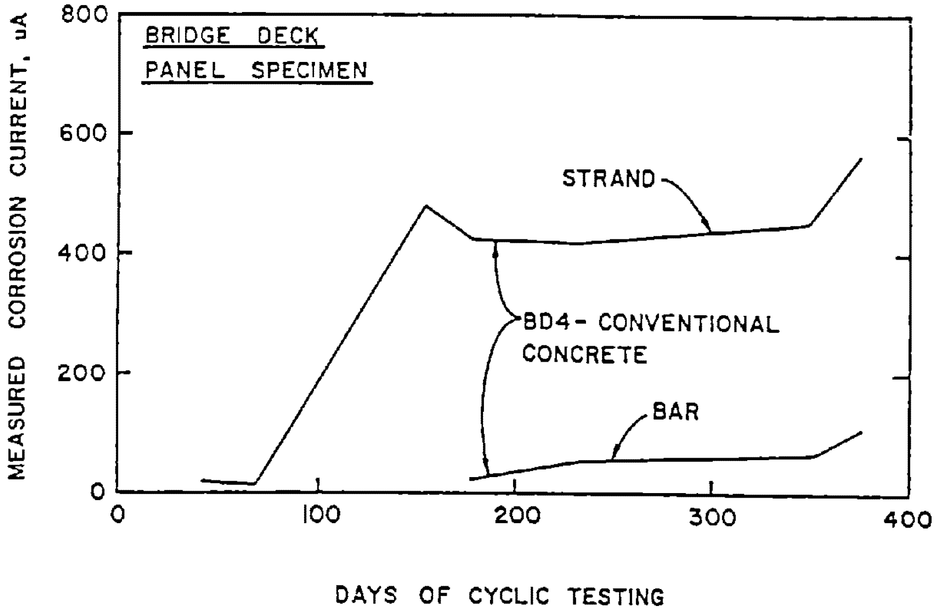


Fig. 19 - Corrosion activity data from prestressed concrete bridge deck panel BD4.

The final average chloride contents at the 1/2- and 1-in. depths were 0.373 and 0.176 percent, respectively. The final chloride content at the 1-in. depth was about 6 times greater than the usual corrosion threshold level. When the two strands were examined, corrosion by-products on their half-perimeter areas covered 21 and 3 percent, respectively. The bar had corrosion by-products on about 6 percent of the same half-perimeter area. While the corrosion activity of BD4 was greater than that for BD1, which had a permanent macrocell connection, this greater corrosion activity is probably related to the higher final chloride content at the 1-in. depth in the BD4 specimen. Both BD1 and BD4 had essentially identical total anode to cathode AC resistance values of 25 to 40 ohms during the test period.

General - In the two unprotected conventional concrete bridge deck specimens and the three unprotected conventional concrete pile specimens, there were 14 pieces of full- or short-length gray bars with 1-in. cover. With the two bridge deck specimens, only one of the two bars in these two specimens showed obvious corrosion by-products. This occurred in spite of the final chloride contents at the bar depth being 2 to 6 times greater than the usual corrosion threshold level. The corroded bar had corrosion by-products on only 6 percent of the half-perimeter area even though the final chloride content was 6 times the threshold level.

With the three unprotected conventional concrete piles, there was a total of 12 pieces of gray bar with 1-in. cover. Eleven pieces showed no obvious corrosion by-products even though the final chloride contents in the wet/dry and ponded regions were about 3 and 6 times the corrosion threshold, respectively.

With the bare prestressing strand, similar trends were observed with the conventional concrete pile specimens P1 and P2. The four gray strands in the crack-free areas of these two piles did not show any obvious corrosion by-products even though the final chloride levels at the strand were 3 to 6 times the corrosion threshold, as discussed above. Similar trends with gray strands in conventional concrete bridge deck specimens BD1 and BD4 were not observed. Three of the four strands developed significant corrosion by-products.

These data suggest that the reinforcing bars in the accelerated heat-cured, precast, prestressed concrete members were much better protected from corrosion, compared to the moist-cured reinforced concrete beams and columns having the same 1-in. cover and 0.44 w/c ratio concrete, even though the final chloride contents at the bars were always far in excess of the threshold level.

Electrical Resistance Properties of Specimens

The electrical resistance of the four specimen types was measured. The average electrical resistance of the concrete/reinforcing steel system from the potentially anodic steel grid to the potentially cathodic steel grid are tabulated below. Values at the beginning and at the conclusion of the cyclic testing are given.

Specimen type	Average AC resistance, ohms	
	Initial	Final
Column - gray bars	35	65
Column - epoxy bars	2,000	18,000
Beam - gray bars	55	100
Beam - epoxy bars	2,300	18,000
Beam - silica fume and gray bars	1,800	5,000
Bridge deck - gray bar and strands	25	40
Piles - gray bars and strands	30	55
Piles - epoxy strands	81,000	100,000

These data show the very beneficial effect of using epoxy-coated bars or epoxy-coated strand or silica fume concrete. These materials provide very high internal electrical resistance compared to the very low electrical resistance of conventional concrete specimens with gray bars or gray strands.

The initial electrical resistance of the silica fume concrete beam specimen was about 33 times greater than the conventional concrete beam specimen. This factor is essentially the same as the 25 times greater factor determined during private industry studies on 7-in. thick slab specimens.

The silane-treated column, beam and bridge deck panel specimens had similar initial and final electrical resistance compared to the same specimen type made with unprotected conventional concrete.

The final electrical resistance values for the conventional concrete columns, beams, bridge deck and piles which were reinforced with gray bars or strands were about 75 percent greater than the initial values. This 75 percent increase is much less than the average 375 percent increase measured on the conventional concrete slabs (System 1) in the pilot studies. The lower increase is attributed to the larger specimen sizes and the periodic fresh water rinsing operations which supplied moisture to all surfaces of these full-size specimens.

Chloride Content Profiles in Concrete After Cyclic Testing

General - The measured individual and average chloride ion contents from the full-size specimens are tabulated in Tables 10 to 13 in Appendix E. The chloride data from the moist-cured reinforced concrete columns and beams are given in Tables 10 and 11. The chloride data from the accelerated heat-cured prestressed concrete piles and bridge deck panels are given in Tables 12 and 13. A summary of the average chloride content data that includes the mean value, the standard deviation and the coefficient of variation of the significant measured chloride data from all 19 specimens is presented in Table 7.

Horizontal versus vertical beam surfaces - A review of the chloride data shows that there are very small to moderate differences in the final average chloride contents from cores taken from saltwater-exposed horizontal or vertical beam surfaces. For the B1, B3 and B4 specimens, which contained the conventional 0.44 w/c ratio concrete, the overall average horizontal and vertical surface chloride contents at the 1/2-in. depth level were 0.535 and 0.532 percent, respectively. These overall average values had coefficients of variation of 7.1 and 8.6 percent for the six samples each, taken from the horizontal and vertical beam surfaces. The overall average horizontal and vertical beam surface chloride contents at the 1-in. depth were 0.285 and 0.240 percent, respectively, with coefficients of variation of 13.8 and 21.3 percent.

Table 7 - Summary of final average chloride content data from full-size specimens.

Specimen description	Chloride sample depth, in.	No. of samples	Mean value, % by wt of concrete	Standard deviation, % by wt of concrete	Coefficient of variation, %
			Normal Concrete—Unprotected		
Columns C1,C2 C4 and C5	1/2	16	0.5708	0.0612	10.7
	1	16	0.3000	0.0654	21.8
	1 3/4	16	0.0269	0.0271	100.7
Beams B1, B3 and B4	1/2	12	0.5334	0.0401	7.5
	1	12	0.2625	0.0495	18.9
	1 3/4	12	0.0107	0.0062	57.9
Piles P1, P2 and P4 (ponded region)	1/2	6	0.3113	0.0448	14.4
	1	6	0.1625	0.0571	35.1
	1 3/4	6	0.0073	0.0038	52.1
Piles P1, P2 and P4 (cyclic regions)	1/2	6	0.2088	0.0218	10.4
	1	6	0.0830	0.0390	47.0
Bridge deck panels BD1 and BD4	1/2	4	0.3465	0.0383	11.1
	1	4	0.1190	0.0666	56.0
			Silane Treated Normal Concrete		
Column C3	1/2	4	0.0265	0.0157	59.2
Beam B5	1/2	4	0.0045	0.0006	13.3
Bridge deck panel BD3	1/2	2	<0.004	—	—
			Calcium Nitrite Admixture in Concrete		
Column C6	1/2	4	0.6083	0.0539	8.9
	1	4	0.3193	0.0496	15.5
	1 3/4	4	0.0155	0.0075	48.4
Pile P3 (ponded region)	1/2	2	0.4455	0.0361	8.1
	1	2	0.2645	0.0120	4.5
Pile P3 (cyclic region	1/2	2	0.4150	0.0396	9.5
	1	2	0.2100	0.0226	10.8
Bridge deck panel BD2	1/2	2	0.4350	0.0085	2.0
	1	2	0.1610	0.0127	7.9
			Silica Fume Admixture in Concrete		
Beam B2	1/2	4	0.1005	0.0469	46.7
	1	4	0.0060	0.0012	20.0
	1 3/4	4	0.0048	0.0005	10.4

These data show that the orientation of the flowing salt water on the beams had no measurable effect on the ingress of chloride to the 1/2-in. depth, while the average chloride ingress to the 1-in. depth was about 20 percent greater into the horizontal surface as compared to the vertical surface.

Upper region versus lower region of columns - Two cores each were taken from the upper and lower regions of each column for comparative purposes. For the C1, C2, C4 and C5 specimens, which contained unprotected conventional concrete, the overall average upper and lower region vertical column surface chloride contents at the 1/2-in. depth were 0.556 and 0.586 percent, respectively. Coefficients of variation were 9.4 and 11.8 percent. The overall average upper and lower region chloride contents at the 1-in. depth were 0.275 and 0.325 percent, respectively, with coefficients of variation of 20.7 and 20.7 percent. These data show that the lower regions have slightly higher chloride contents than the upper regions although the differences are small, i.e., from 5 to 18 percent.

Moist-cured conventional concrete columns and beams - The overall average chloride content profiles from columns C1, C2, C4 and C5 and beams B1, B3 and B4 are shown in Fig. 20. The plotted data from the columns consisted of 16 samples for each depth. For the beams, each plotted data point consisted of 12 different samples for each depth. The value at the 3 1/4-in. depth level was not plotted because its chloride value was essentially zero.

The data in Fig. 20 show that the columns and beams have essentially the same chloride profiles. These average data show that the chloride ion corrosion threshold level of approximately 0.03 percent by weight of concrete was exceeded where the clear cover was about 1 1/2 to 1 3/4 in. At the 1-in. depth, the chloride contents from either specimen type was about 10 times the corrosion threshold level and at the 1/2-in. depth, the chloride contents were about 18 times the corrosion threshold level.

These chloride data from a high-quality, 0.44 w/c ratio, moist-cured concrete having a 28-day compression strength of almost 6000 psi show that chloride was able to penetrate into the concrete through 1 1/2 to 1 3/4 in. of cover to equal or exceed the chloride corrosion threshold level. The data also show that the chloride contents from the 1 3/4-in. to the 3 1/4-in. depth levels were extremely low, generally less than the corrosion threshold level, and essentially zero in the 2 1/2 to 3 1/4 in. depth increment.

Moist-cured calcium nitrite admixture concrete column - The overall average chloride content profile from column C6 is shown in Fig. 21 as compared to the four conventional concrete columns. The data show that the chloride profile in the calcium nitrite Column C6 is essentially identical to conventional concrete although the concrete made with calcium nitrite showed somewhat higher chloride contents.

Moist-cured silane-treated columns and beams - The overall average chloride content profiles from column C3 and beam B5, which were silane-treated conventional concretes, are shown in Fig. 22. Results for unprotected conventional concrete columns and beams are also shown. These data show that the silane treatment essentially prevented any ingress of chloride-laden water from the 1-in. to the 3 1/4-in. depth levels. At the 1/2-in. depths, the B5 specimen also contained essentially zero chloride. The C3 specimen contained at the 1/2-in. depth an average of 0.027 percent chloride, a value less than the corrosion threshold.

These data show that at the 1/2- and 1-in. depth levels, the silane treatment prevented the ingress of saltwater by about 95 to 99 percent and by about 98 percent, respectively, compared to unprotected conventional concrete columns and beams.

Moist-cured silica fume admixture concrete beam - The overall average chloride content profile from beam B2 is shown in Fig. 23 as compared to

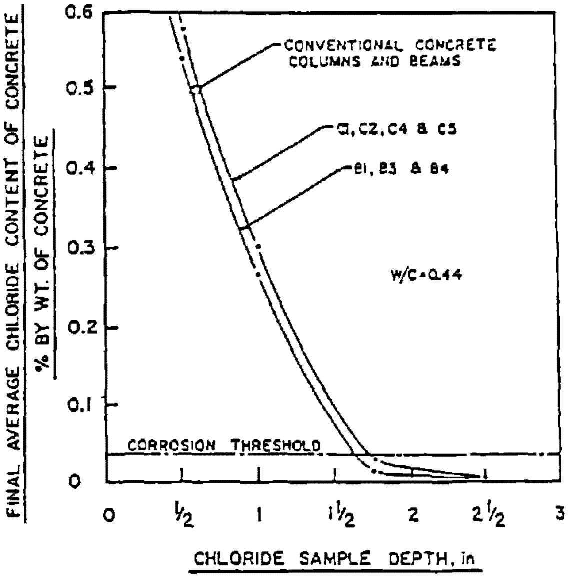


Fig. 20 - Chloride profiles for conventional concrete columns and beams.

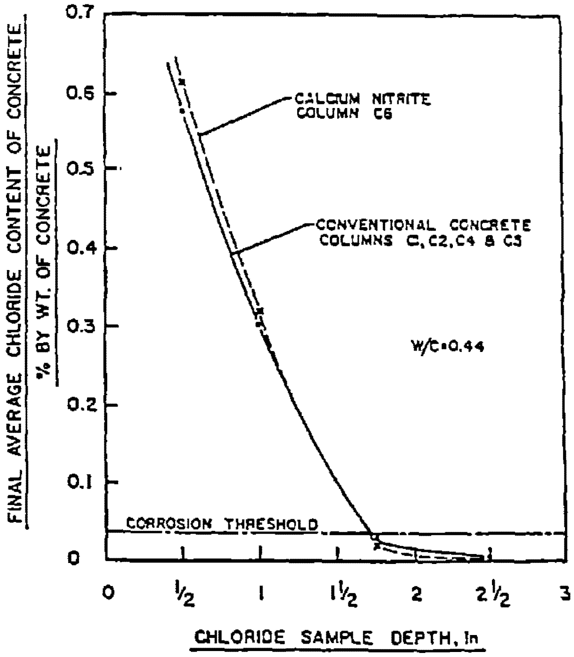


Fig. 21 - Chloride profiles for calcium nitrite and conventional concrete columns.

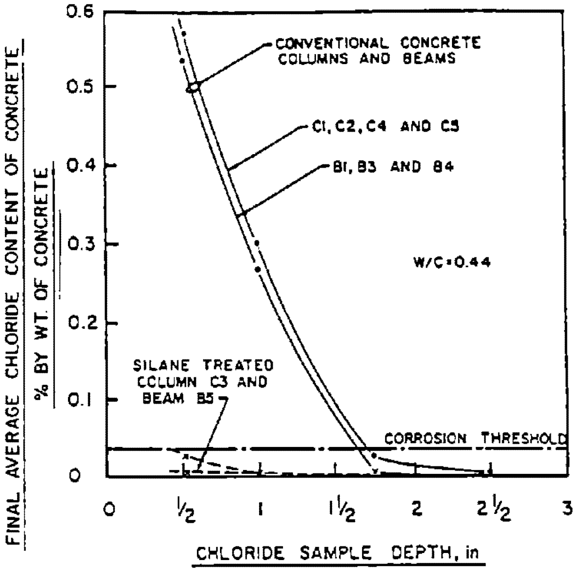


Fig. 22 - Chloride profiles for silane-treated and conventional concrete columns and beams.

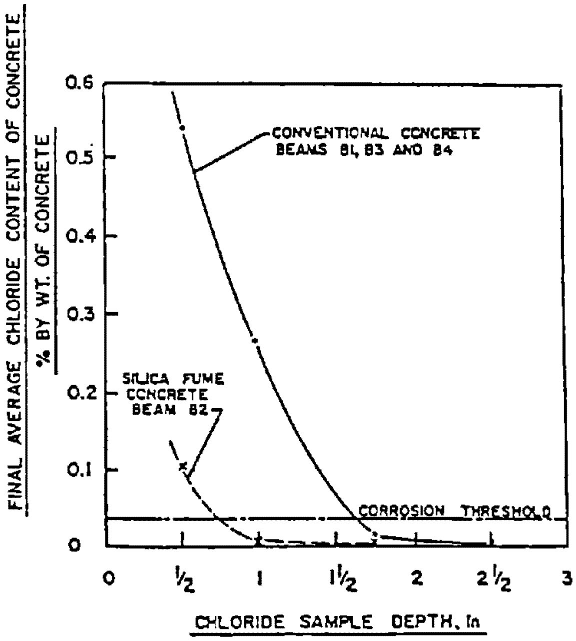


Fig. 23 - Chloride profiles for silica fume concrete and conventional concrete beams.

the three conventional concrete beams. These data show that the silica fume concrete prevented any ingress of chloride-laden water from the 1-in. to the 3 1/4-in. depths. At the 1/2-in. depth, the B2 specimen contained an average of 0.100 percent chloride ion, a value about 2 to 3 times the corrosion threshold.

These data show that at the 1/2- and 1-in. depths, the silica fume concrete reduced the ingress of saltwater by about 80 and 98 percent, respectively, compared to the three unprotected conventional concrete beams.

Heat-cured prestressed conventional concrete bridge deck panels - The bridge deck panels BD1 and BD4 contained the same 0.44 w/c ratio conventional concrete as the columns and beams. However, these bridge deck panels were heat-cured overnight at about 140°F. They were then uniformly prestressed. The cyclic testing consisted of the same 4 hr per day exposure to flowing saltwater as the columns and beams.

The overall average chloride content profile for BD1 and BD4 is shown in Fig. 24, as compared to the chloride profiles from the moist-cured, reinforced concrete columns and beams. The plotted data for the bridge deck panels consisted of four samples for each depth.

These data show that the heat-cured, prestressed concrete bridge deck panels absorbed about 40 and 60 percent less chloride at the 1/2- and 1-in. depths, respectively, compared to the moist-cured columns and beams. This comparison shows that the overnight heat-cured concrete is about 50 percent less permeable to chloride-laden water, compared to the seven concrete beam and column specimens which were cured in their forms under moist burlap conditions for 3 days. While the chloride absorption was much less than moist-cured concrete, the chloride levels still exceeded the corrosion threshold level at about 1 1/2 in. of cover.

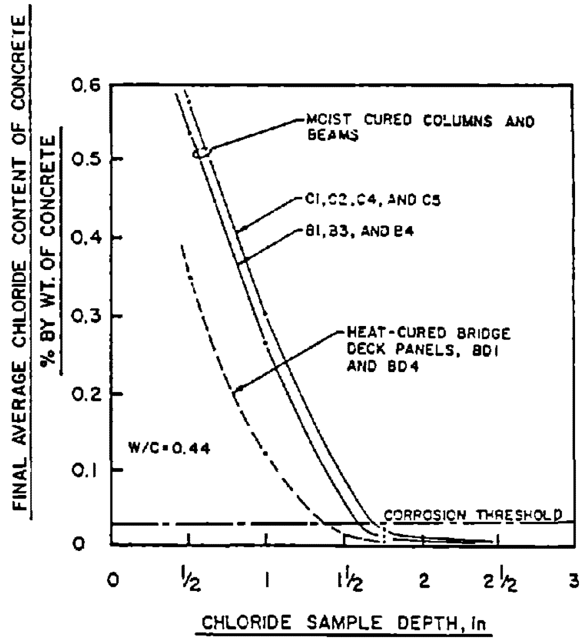


Fig. 24 - Chloride profiles for conventional concrete specimens subjected to 4 hr per day saltwater flow.

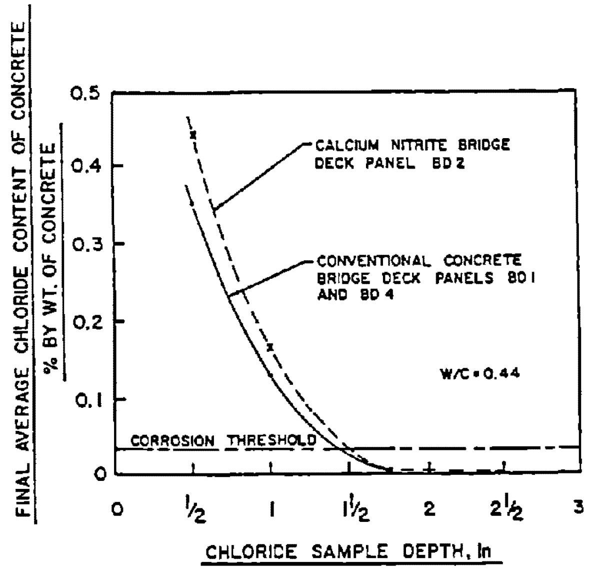


Fig. 25 - Chloride profiles for calcium nitrite and conventional concrete bridge deck panels.

Heat-cured, prestressed calcium nitrite concrete bridge deck panel - The bridge deck panel BD2 utilized the same 0.44 w/c ratio concrete but with the calcium nitrite admixture. This specimen was also heat-cured overnight at about 140°F and then prestressed.

The overall average chloride content profile for BD2 is shown in Fig. 25, as compared to the average chloride profile from the two conventional concrete bridge deck specimens BD1 and BD4. These data show that the chloride contents of the calcium nitrite concrete bridge deck panels are about 25 to 30 percent higher at the 1/2- and 1-in. depths, respectively, compared to the conventional concrete bridge deck panels.

These data also show that the BD2 heat-cured bridge deck chloride contents at the 1/2- and 1-in. depths are about 30 and 50 percent less, respectively, than the chloride contents in the moist-cured calcium nitrite column C6. This comparison shows that the overnight heat-cured calcium nitrite concrete is about 40 percent less permeable to chloride-laden water, compared to a similar specimen which was cured in a form under moist burlap conditions for 3 days. This 40 percent reduction in permeability due to heat curing is similar to the average 50 percent reduction found with conventional concretes. Thus, heat curing benefically reduces chloride penetration when compared to 3-day moist-cured reinforced concrete.

Heat-cured, silane-treated, prestressed concrete bridge deck panel - Bridge deck panel BD3 utilized silane-treated conventional 0.44 w/c ratio concrete. This heat-cured, prestressed panel was silane-treated in the same manner as the moist-cured column C3 and beam B5.

The chloride data, as shown in Fig. 26, show that the silane treatment stopped 99 percent of the ingress of chloride to the 1/2 in. or greater depth, when compared to the chloride contents in the unprotected, conventional concrete BD1 and BD4 specimens.

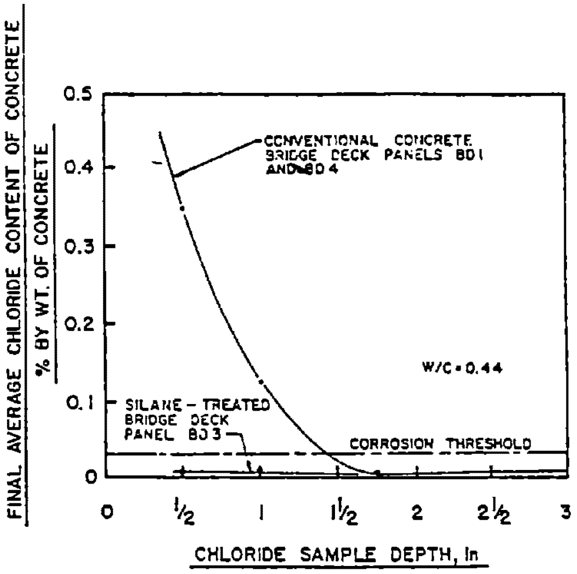


Fig. 26 - Chloride profiles for silane-treated and conventional concrete bridge deck panels.

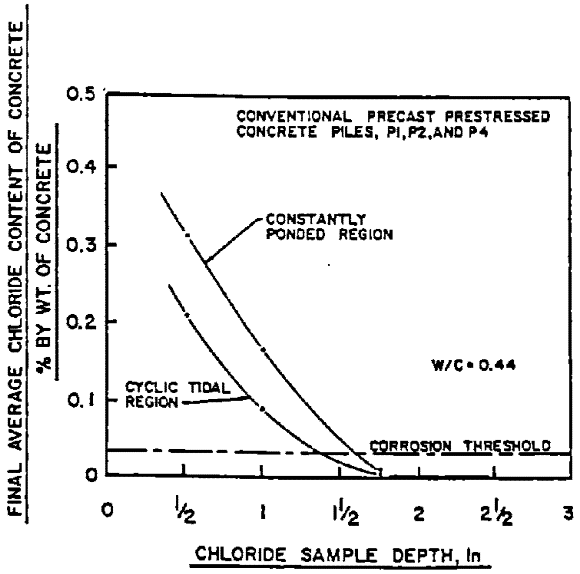


Fig. 27 - Chloride profiles for conventional concrete piles.

Heat-cured, prestressed, conventional concrete piles - The pile specimens were exposed to continuous saltwater ponding for about 370 days in the simulated below-water test region. The other saltwater-exposed simulated tidal zone test region was cycled on a continuous 6-hr basis, from flowing saltwater to air drying. Thus, these piles were exposed to about 8900 hrs of constant saltwater ponding in one test area and about 4450 hrs of cyclic saltwater exposure in the other test area. These exposure times are much greater than the 1500 hrs of total exposure to flowing saltwater that the heat-cured precast bridge decks experienced. However, as the chloride data show, the 4 hr per day exposure that the bridge decks experienced produced the greatest ingress of chloride ions to the 1/2- and 1-in. depths, compared to the cyclic region in the piles. In fact, the final chloride contents at the 1-in. depth were about 50 percent greater in the heat-cured bridge deck panels, compared to the cyclic area chloride contents from the three conventional concrete piles. This greater severity is attributed to the longer, 20 hr per day air drying period that the bridge decks experienced.

The overall average chloride content profiles from piles P1, P2 and P4 which utilized the conventional 0.44 w/c ratio concrete are shown in Fig. 27. The chloride contents at the 1/2- and 1-in. depth are about 50 and 100 percent greater, respectively, in the ponded region compared to the cyclic region.

While the chloride contents of these heat-cured, prestressed concrete piles are lower than moist-cured columns and beams, the final chloride contents are still in excess of the usual 0.03 percent corrosion threshold level at a depth of about 1 1/2 in.

Heat-cured, prestressed calcium nitrite concrete pile - Pile P3 contained the calcium nitrite admixtured concrete at the same 0.44 w/c ratio as the conventional concrete piles P1, P2 and P4. The overall average chloride content profiles for P3, as compared to P1, P2 and P4 are shown in Fig. 28 for the constantly ponded region and the cyclic tidal region. These data show that the calcium nitrite concrete absorbed more chloride than the conventional concrete piles, irrespective of the test region exposure. The increase in chlorides at the 1-in. depth in the cyclic tidal region was about 150 percent, and in the constantly ponded region the increase was about 60 percent. These increases in chloride absorption in the calcium nitrite concrete pile specimen compare with a similar 30 percent increase at the 1-in. depth in the bridge deck panel containing calcium nitrite.

Half-Cell Potential and Corrosion Current Relationships for Full-Size Specimens

The pilot study data from the conventional concrete System 1 slabs (1-in. cover and 0.50 w/c ratio concrete) and companion identical slabs from a concurrent private industry study showed that there is a relationship between macrocell corrosion current and half-cell potential. This linear relationship is shown in Fig. 6. This same linear plot is shown in Fig. 29 labeled as System 1 slabs. The half-cell potential and corrosion current data from the System 6 calcium nitrite concrete pilot study slabs were analyzed. The resultant linear regression analyses plot for the 35 data points from these calcium nitrite slabs with 1-in. cover and 0.50 w/c ratio are also shown in Fig. 29. These two linear plots for System 1 and 6 slabs intercept the half-cell potential axis for zero corrosion current at -0.238 and -0.225 volts, respectively. These data show that conventional concrete and calcium nitrite concrete slabs both start to exhibit corrosion activity at half-cell potentials of about -0.23 volts. The subsequent development of corrosion activity for both System 1 and 6 concretes is similar, as shown in Fig. 29, up to a half-cell potential of about -0.40 volts. However, the calcium nitrite concrete eventually develops much less corrosion current and corrosion by-products, as previously discussed.

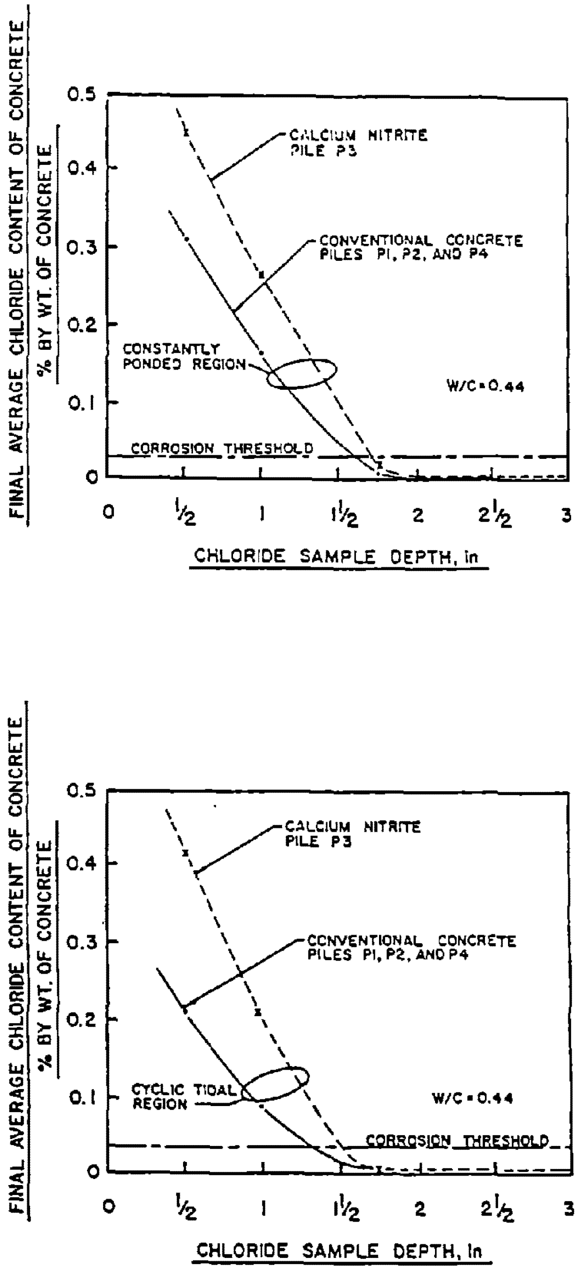


Fig. 28 - Chloride profiles for calcium nitrite and conventional concrete piles.

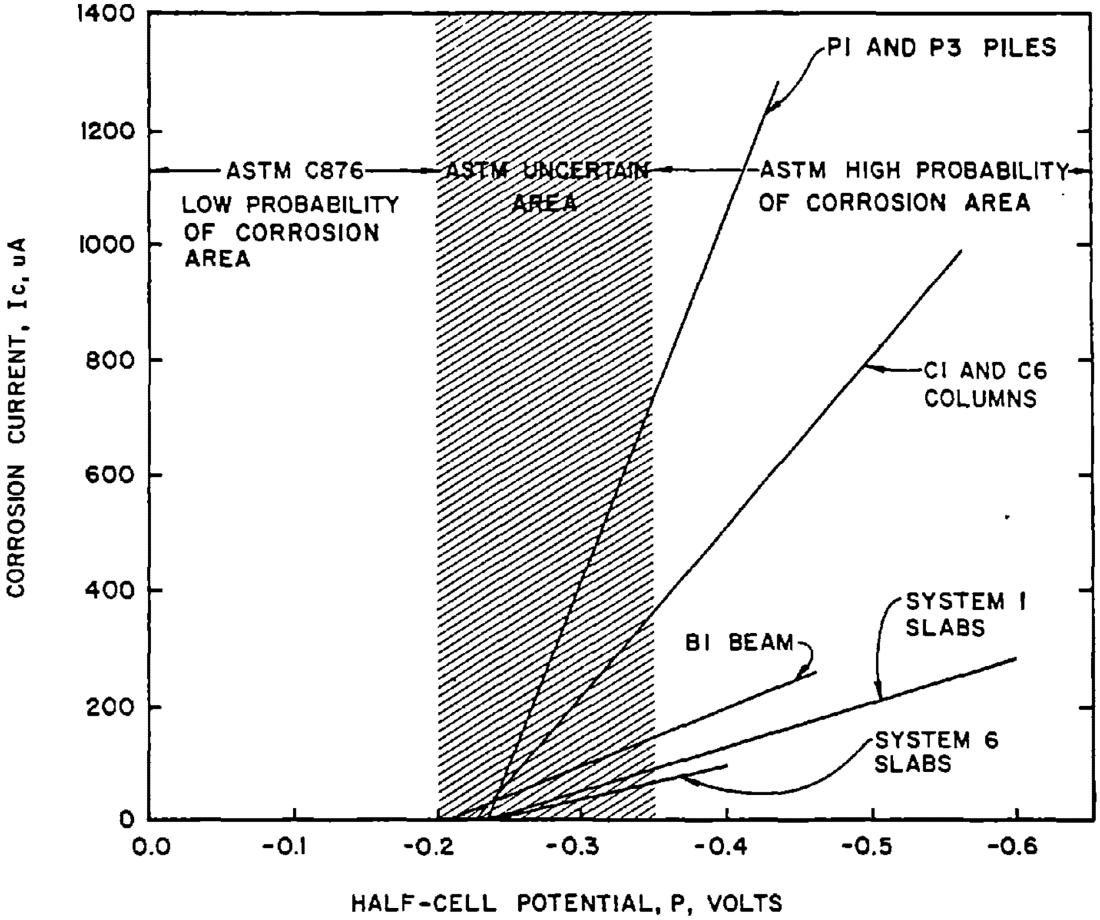


Fig. 29 - Relationship between corrosion current and half-cell potential for slabs, beams, columns and piles.

The corrosion current and half-cell potential data from five of the full-size specimens that corroded have been analyzed to determine if similar linear trends exist between these two measures of corrosion activity. The moist-cured conventional concrete beam B1 was analyzed in a similar linear manner. Ten data points were utilized. The intercept on the half-cell potential axis as shown in Fig. 29 is at -0.208 volts. The data from the moist-cured conventional concrete column C1 and the calcium nitrite concrete column C6 were analyzed together. A total of 17 data points were utilized and the intercept is at -0.227 volts as shown in Fig. 29. The heat-cured conventional concrete pile P1 and the calcium nitrite concrete pile P3 were also analyzed together. A total of 13 data points from crack-free and cracked areas in these two piles were utilized. The resultant intercept is at -0.238 volts as shown in Fig. 29.

The straight-line linear regression equations for these various slabs, columns, beams and pile specimens are tabulated below.

Specimen type	Linear regression equation	Correlation coefficient
System 1 slabs	$I_c = -774.2\ P-184.2$	0.92
System 6 slabs	$I_c = -554.4\ P-124.5$	0.85
Columns C1 and C6	$I_c = -2922.7\ P-662.0$	0.98
Beam B1	$I_c = -1015.0\ P-210.9$	0.75
Piles P1 and P3	$I_c = -6530.7\ P-1553.7$	0.90

The plotted data in Fig. 29 show that these various specimens with different sizes, reinforcing steels, concrete types, curing procedures, ratios of anodic to cathodic steel areas, and saltwater exposure conditions exhibit reasonably linear relationships between corrosion current and half-cell potentials. The correlation coefficients for the data from these five different specimen types ranged from 0.75 to 0.98. The weighted average is 0.91. The intercept on the half-cell potential axis at zero corrosion current ranged from -0.208 to -0.238 volts and averaged -0.227 volts, with a coefficient of variation of only 5.4 percent.

These data show that corrosion activity for all these various specimens started when the half-cell potential exceeded about -0.23 volts. In addition, there is a linear relationship between the subsequently developed corrosion current and half-cell potential, although this relationship is dependent upon many factors.

The ASTM C876 half-cell potential regions of "low" and "high" probability of active corrosion and the "uncertain" region are also shown in Fig. 29. The measured data from this study show that the "low probability" region was an area with no measurable corrosion current, the "high probability" region was an area with significant measurable corrosion current, and the "uncertain" region was the region where corrosion activity initiated and began to increase in magnitude.

The impact of this finding is that in interpreting half-cell surveys on bridges and bridge members, corrosion is probably underway at half-cell potentials of -0.23 to -0.35 volts.

SUMMARY AND CONCLUSIONS

Summary of Investigation

A comprehensive three-year corrosion research project on eleven different corrosion protection systems was undertaken in two laboratory studies. These studies focused on reinforced concrete bridge substructure members and precast, prestressed concrete bridge members. A total of 124 small reinforced concrete slabs were subjected to 48-week, cyclic wet and dry saltwater exposure tests in the first study. The second year-long study dealt with similar cyclic saltwater exposure tests on 19 full-size sections of reinforced concrete columns and beams and precast, prestressed piles and stay-in-place bridge deck panels.

The slab tests evaluated concrete having w/c ratios of 0.51, 0.40 and 0.28 with clear cover of 1, 2, and 3 in. over the embedded steel. The reinforcing steels evaluated were normal gray bars, normal prestressing strands, galvanized bars, and fusion bonded epoxy-coated bars and prestressing strands. A calcium nitrite corrosion-inhibiting admixture for fresh concrete was evaluated. A penetrating silane sealer and a methacrylate coating system were evaluated, as surface treatments for hardened concrete. The full-size member tests evaluated most of the above mentioned materials at a constant AASHTO-quality w/c ratio of 0.44, generally with 1-in. clear cover. A concrete containing a silica fume admixture was also evaluated in the full-size member tests. The full-size columns and beams were moist cured while the precast, prestressed piles and bridge deck panels were heat cured overnight at 130 to 140°F.

Corrosion activity was determined by measuring macrocell corrosion current and instant-off potential between the potentially anodic and cathodic reinforcing steel systems. Half-cell potential of the reinforcing steel in the anodic or corroding region was also measured, as was the electrical resistance of the specimens between the potentially anodic and cathodic steel layers.

Chloride contents were measured in the slab studies when a surge in corrosion current was first observed. This defined the chloride ion corrosion threshold value. Chloride content profiles were also determined at the end of both studies after the extensive cyclic testing, to define the effects of concrete w/c ratio and clear cover on the long-term ingress of chloride into these various concrete specimens.

The corroded reinforcing bars and prestressing strands were examined at the end of the cyclic testing to determine the extent of corrosion.

Conclusions of Investigation

The following conclusions and observations are based upon the data presented in this report as well as in other corrosion-related reports referenced in this report.

- Long-term chloride permeability to the 1-in. depth level was reduced by about 80 percent when the w/c ratio was lowered from 0.51 to 0.40. The reduction was about 95 percent when the w/c ratio was further reduced to 0.28. Accompanying increases in 28-day compressive strengths for these two lower w/c ratio concretes were only 20 and 50 percent, respectively, compared to the compressive strength of the 0.51 w/c ratio concrete.

- When corrosion began under a given set of conditions, the subsequent severity of the corrosion process was reduced significantly when lower w/c ratio concretes were tested. This observation was made on both conventional concrete and concrete specimens containing calcium nitrite.

- Early corrosion of gray bars occurred in the moist-cured concrete when the clear cover was 1 in., irrespective of the w/c ratio. The exception to the above was the silica fume admixture concrete, which did not develop corrosion with 1-in. cover. These data, as well as previous FHWA data show that 1-in. clear cover is not adequate protection against chloride-induced corrosion with w/c ratios ranging from 0.60 to 0.28.

- The various conventional concrete specimens which had 1 1/2, 2 or 3 in. of cover over gray bars or strands did not develop any corrosion during either of these two long-term studies.

This behavior was observed at all investigated w/c ratios. This total lack of corrosion can be attributed to the insufficient ingress of chloride-laden water to these deeper regions. With all investigated w/c ratios, the final chloride contents at the level of the steel at these deeper locations were less than the usual corrosion threshold of about 0.03 percent by weight of concrete. Thus, these two, year-long investigations both show that corrosion will be delayed considerably when a reasonable clear cover is used. For the current AASHTO 0.44 w/c ratio concrete quality, these tests show that the typically specified 2-in. clear cover provided very good corrosion protection by keeping the final chloride content at the 2-in. depth less than the corrosion threshold level. As a comparison, the typical final chloride contents at the 1-in. depth were 15 to 20 times that at the 2-in. depth for the 0.51, 0.44, and 0.40 w/c ratio, moist-cured concretes evaluated in these two studies. With the moist-cured 0.28 w/c ratio concrete, the final chloride content at the 1-in. depth was only two times that at the 2-in. depth. With the heat-cured precast, prestressed concretes at the 0.44 w/c ratio, the typical final chloride contents at the 1-in. depth were 20 to 30 times that at the 2-in. depth.

- The accelerated heat-cured, precast, prestressed concrete members absorbed about 30 to 50 percent less chloride in the first 1 in. of concrete compared to the moist-cured concrete members. This was observed for conventional as well as calcium nitrite concretes, all having the 0.44 w/c ratio. These data suggest that the permeability of the heat-cured AASHTO-quality concrete is measurably lower than that of three-day moist-cured concrete. While the heat-cured concrete members absorbed much less chloride, their final chloride contents still exceeded the usual corrosion threshold level at depths of about 1 1/2 to 1 3/4 in. Thus, the 2-in. clear cover condition was still generally required to achieve adequate protection under these test conditions.

- Fusion bonded epoxy-coated bars and prestressing strands did not develop any corrosion activity in either study, even with 1-in. cover. The above conclusion applies when both reinforcing mats contained epoxy-coated steels or when only the potentially anodic mat contained epoxy-coated steel while the potentially cathodic mat contained normal gray bars. Final chloride contents adjacent to these coated steels were as high as 20 times the corrosion threshold level. Holidays, premarked on the coated deformed bars, did not develop any corrosion. The coated prestressing strand did not contain any holidays at all. Specimens made with coated bars or strands produce very high internal electrical resistance due to the properties of the epoxy-coating. The specimens made with coated strands have exceptionally high electrical resistance values since there are no holidays in the coating, which is also much thicker than the coating on deformed bars.

- The calcium nitrite corrosion-inhibiting admixture did not significantly delay the initiation of corrosion compared to conventional concrete specimens during both studies. However, the severity of the subsequent corrosion process (total amp-hours of corrosion current) on gray bars and strands was reduced significantly when compared to conventional concrete specimens. The amount of corrosion by-products on the gray bars was about one-tenth that measured on bars from conventional concrete specimens from both studies. The chloride permeability of the calcium nitrite concrete was similar to conventional concrete when moist-cured, but it was greater when heat-cured.

- Galvanized bars and gray bars, when used in the same specimen, developed very large and sustained corrosion currents. The measured corrosion activity was similar to specimens containing only unprotected gray bars. Zinc corrosion by-products and small amounts of steel corrosion by-products were observed on the galvanized bars, which indicated that the zinc coating was corroded away in small, localized areas. This was confirmed by microscopic examinations on cut and polished cross-sections.

- Galvanized bars, when used exclusively within these various specimens, developed very low corrosion currents. The measured corrosion activity was similar to or less than that measured on the calcium nitrite concrete specimens, which utilized gray bars. Zinc corrosion by-products and extremely small amounts of steel corrosion by-products eventually developed on the galvanized bars.

- Silane-treated conventional concrete specimens, moist-cured and heat-cured, did not develop corrosion activity, except for minor corrosion in one slab specimen. None of the full-size specimens developed any corrosion activity. The silane treatment reduced the ingress of chloride to the 1/2- and 1-in. depths by about 95 to 99 percent compared to unprotected conventional concrete specimens. The final chloride contents at the level of the gray bars and strands were less than the corrosion threshold level.

- The methacrylate-coated conventional concrete specimens developed significant corrosion activity, with 1- and 2-in. cover. Chloride content tests were highly variable at initiation of corrosion and after the cyclic tests. The electrical resistance of these coated specimens was also highly variable. These large differences in behavior between specimens suggest non-uniform protection. The coated surfaces showed evidence of coating disintegration at random locations at the end of the cyclic testing. Coating deterioration may explain the non-uniform behavior. Further testing should be undertaken to verify this behavior which does not agree with previous NCHRP data.(17)

- Steel in slabs which contained silica fume admixture concrete, with 1-in. cover over gray bars, did not develop any corrosion. This was the only moist-cured, unprotected concrete specimen with 1-in. cover over gray bars which was corrosion free. The chloride contents at the 1-in. or greater depths were reduced by about 98 percent compared to conventional 0.44 w/c ratio moist-cured concrete. The final chloride content at the level of the gray bars was less than the corrosion threshold level.

- Corrosion of gray bars in moist-cured concrete began when the acid-soluble chloride content at the level of the bars reached about 0.18 to 0.26 percent by weight of portland cement. These threshold values correlate well with previously reported threshold levels. These data suggest that a realistic design corrosion threshold level is 0.20 percent for moist-cured concrete. For some unknown reason, the gray bars in the heat-cured, precast, prestressed concrete piles and bridge deck members generally did not develop any macrocell corrosion activity. In fact, 85 percent of these gray bars in heat-cured, precast, prestressed conventional concrete piles and bridge deck panels did not develop significant corrosion by-products even though the final chloride contents were two to six times the corrosion threshold level. With the full-size moist-cured columns and beams, the potentially anodic gray bars all developed corrosion activity and corrosion by-products. These data suggest that gray reinforcing bars in heat-cured, precast, prestressed concrete members were better protected from chloride-induced corrosion, compared to gray bars in moist-cured reinforced concrete columns and beams having the same 1-in. cover of 0.44 w/c ratio concrete.

- Corrosion activity of unstressed gray prestressing strands with 1-in. cover in moist-cured concrete began when the chloride content at the strand level reached an average of almost 1.2 percent by weight of portland cement. This unusually high corrosion threshold level is six times greater than that determined for gray bars. In addition, the average time-to-corrosion for these unstressed prestressing strands was three times as long as that measured for gray bars. During the tests of full-size members, which used fully-stressed prestressing strands, the strands did not exhibit the same longer time-to-corrosion behavior compared to gray bars. In fact, the gray strands generally showed corrosion activity prior to the gray bars, both with 1-in. cover. However, over 60 percent of the fully-stressed gray strands in the four heat-cured, precast, prestressed, conventional concrete piles and bridge deck members showed no significant corrosion by-products on the strands even though the final chloride contents adjacent to these strands ranged from two to six times the usual corrosion threshold. The other 40 percent had corrosion by-products ranging from 3 to 21 percent of the strand half-perimeter area.

- At initiation of corrosion activity, small slabs and full-size members with gray bars or strands developed significant and simultaneous increases in macrocell corrosion current, instant-off potential, and half-cell potential. Once significant corrosion currents began, the data from these two studies show that there is a reasonably linear relationship between corrosion current and half-cell potential. The data show that corrosion currents began when the ASTM C876 half-cell potential exceeded about -0.23 volts. The above conclusions were made on small- and full-size conventional and calcium nitrite concrete specimens. The impact of this finding is that in interpreting half-cell surveys on bridges and bridge members, corrosion is probably underway at half-cell potentials of -0.23 to -0.35 volts. This range of potential is within the voltage limits referred to in ASTM C876 as the "uncertain" region.

- The gradual ingress of chloride-laden water into uncracked 5000 to 7500 psi air-entrained concrete made with 0.51 to 0.28 w/c ratios and 1-in. clear cover can cause corrosion of the embedded gray bars or prestressing strands. This corrosion process can eventually cause cracking in the concrete adjacent to the corroded steel.

Design Considerations

This section offers suggestions for producing corrosion-resistant new substructure concrete members and new precast, prestressed concrete members for bridges. They are based upon the conclusions of this investigation as well as conclusions from other corrosion-related reports referenced in this report.

Water-cement ratio of concrete - Concrete w/c ratio should be as low as possible, preferably in the 0.44 to 0.32 range. The chloride permeability of the concrete and the severity of corrosion, if it occurs, will both be decreased as the w/c ratio is decreased.

Clear cover - Clear cover over reinforcement should be as large as possible to minimize the long-term penetration of water and chloride ions to the steel. The shape of the chloride content profiles at the end of these two cyclic test programs, and those reported by FHWA(12), for w/c ratios of 0.44 or less show that the risk of long-term corrosion is increased when the actual clear cover is less than 1 3/4 in. Since the actual clear cover does not include the 1/2 in. additional clear cover(12) usually required to account for construction tolerances in bar positioning, an increased risk of early corrosion is associated with design clear covers of 2 1/4 in. or less for usual bridge members.

Use of coated bars or coated prestressing strands - While these two studies and other FHWA studies utilized initially uncracked concrete specimens, the advantage of coated bars or coated prestressing strands in providing corrosion protection is apparent in concrete bridge members which contain cracks through which chloride solution can easily reach the steel reinforcing.

Fusion bonded epoxy-coated bars and prestressing strands were corrosion-free at the end of these two studies at all w/c ratios and all clear covers. These results show that when minimum clear cover is allowed, the epoxy-coated steel provides for a corrosion-resistant reinforcing material. However, based upon chloride ingress characteristics into concrete and the possibility of imperfections in the coatings, it is recommended that concrete w/c ratio should be 0.44 or less to further reduce the risk factor.

The use of effective concrete surface sealers can provide significant additional corrosion protection for members containing coated steels.

Other means of providing corrosion protection for normal gray bars or prestressing strands - The use of calcium nitrite or other effective corrosion-inhibiting chemical admixtures can provide significant protection by chemically reducing the severity of the corrosion process when uncoated steel is used. Concrete w/c ratio should be in the 0.32 to 0.44 range to obtain the maximum benefit, and the risk of corrosion is increased when the design clear cover is 2 1/4 in. or less.

The use of highly-effective concrete surface sealers, such as the silane sealer evaluated in these two studies, can provide significant protection by drastically reducing the ingress of any chloride to depths of 1 in. or more. This very good performance could suggest that the clear cover might be reduced when these particular sealers are used. However, it is recommended that the same w/c ratio and clear cover requirements should be utilized when such treatments are to be used on members utilizing gray bars or strands.

A particular very high-strength silica fume concrete mixture showed very good corrosion protection capabilities. Chloride penetration was extremely low. Another advantage is the very high electrical resistivity of this concrete mixture, which should drastically reduce corrosion currents in the event that chloride levels at the reinforcement become high enough to initiate corrosion.

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APPENDIX A

DESCRIPTION OF ELECTRICAL CORROSION-MEASURING SYSTEMS USED IN STUDY

General

The intent of the electrical measurements made on the reinforcing steel was to identify the start of corrosion activity of the steel reinforcement or the zinc coating and to follow the subsequent corrosion activity by three independent test procedures.(19,23) These were:

1. Measuring corrosion current between the upper and lower mats.
2. Determining the instant-off potential or the back voltage (EMF) developed by the corrosion macrocell immediately after the external connection between the upper and lower mats of steel was interrupted.
3. Determining the copper-copper sulfate half-cell potential of the upper mat of steel.

Specific details are described below.

Measured Corrosion Current

Corrosion currents are caused by electron flow which occurs during the corrosion process. They are directly related to the rate at which corrosion is occurring. An external electrical connection through a 10-ohm resistor between the potentially anodic and cathodic steel reinforcing closed the circuit and facilitated the flow of macrocell corrosion currents between the anodic and cathodic steel mats.

A possible method of measuring currents without disturbing current flow is to measure the IR voltage drop across the resistor and calculate corrosion using the basic Ohm's law relationship. The disadvantage of that procedure was that voltage drops across the resistor were so small that a very accurate microvoltmeter is required. An alternative procedure was adopted. This procedure used an analog micro-ammeter driven by an operational amplifier in a conventional "voltage to current" conversion mode. Current measurements were generally made with this meter in series with a 10-ohm resistor. It was capable of measuring currents from the mid-nanoampere range through the microvolt range and had an impedance which was essentially zero. Periodic checks of ammeter readings were made using a microvoltmeter and calculating corrosion values from the data obtained by using Ohm's law. Microcell currents, between different points on the same anodic bar, were not measured in this project.

Instant-Off Potentials

The measurement of the corrosion cell back voltage between the potentially anodic and cathodic steel reinforcing should be made immediately after the external connection is broken. Otherwise, error will occur because the back voltage changes quickly once the connection is broken.

An electronic circuit was built which interrupted the connection between the anodic and cathodic steel for a period of 31 milliseconds. During this disconnect period, potentials were monitored with a digital voltmeter set to a "peak hold" mode. After the off-period, the electronic switch connecting the steel reinforcing was turned on and the peak potential attained during the 31 milliseconds was retained. The 31 millisecond circuit interruption time was arbitrary and reflected a realistic minimum period required for the peak hold mechanism to stabilize.

Fig. 30 shows "instant-off potential" versus time after circuit turnoff for a typical slab with significant corrosion currents.

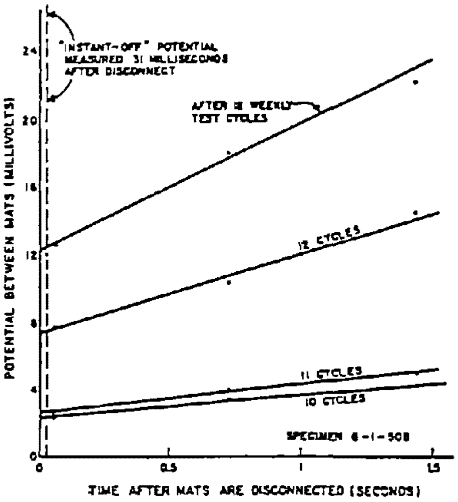


Fig. 30 - Plot of between-mat potential changes after time of disconnect.

Half-Cell Determinations

Copper-copper sulfate half-cell measurements conforming to ASTM C876(23) were determined routinely. The potentials were obtained at the same locations and the readings averaged to obtain the values given in this report.

Concrete Electrical Impedance (Resistance)

The impedance of the electrical path through concrete between the potentially anodic and cathodic steel reinforcing was determined with an AC bridge. The bridge had a frequency of approximately 92 Hz. A variable capacitor was not incorporated in the circuit during routine measurements, consequently the minor correction sometimes required for capacitive effects was not made.

The macrocell corrosion current (I), instant-off potential (V), and concrete impedance (R) were related by the Ohm's law equation $V = IR$.

APPENDIX B

DETAILS OF REINFORCING STEELS, CONCRETE MIXTURES, SPECIMEN FABRICATION, CHLORIDE ION TESTS, AND COATING DETAILS FOR PILOT TIME-TO-CORROSION STUDY

Reinforcing Steel Details

The reinforcing steel bars and prestressing strands were obtained from commercial sources in Illinois, Maryland, Florida, and Indiana. Suppliers stipulated that all these materials met their respective ASTM specifications.

The gray No. 4 bars (Grade 60) were cut, washed in solvent to remove dirt and grease, and air dried prior to epoxy coating their ends.

The 197 fusion bonded epoxy-coated No. 4 bars were tested for holidays with a DC detector and for the coating thickness. The results are as follows:

Number of holidays per ft.	Number of bars	Average coating thickness (mils)	Coating thickness standard deviation (mils)
0	89	9.6	2.00
1	46	9.7	1.76
2	31	9.6	2.09
3	9	7.9	1.68
4	22*	8.4	2.77

*Caused by long scratches, not individual holes

As shown, 45 percent had no holidays, 39 percent had one or two holidays per foot, and 16 percent had three or more holidays per foot. The test slabs utilized only bars which met the ASTM C3963-81(25) limit of no more than two holidays per foot. Each holiday was identified with ink for reference at the end of the testing.

With the epoxy-coated strands, the mil thickness measuring gage would not measure the coating thickness so the coating thickness was measured using microscopic techniques. Ten coated and ten bare 1/2-in. strands were accurately weighed and measured for length. The cut cross-section of the ten coated strands then was carefully examined and measured. The coated strands were also tested for holidays. There were none. The weight data follow:

	Wt. of strand, lbs/ft
Epoxy coated	0.555
Uncoated	0.520

The standard weight of uncoated low relaxation 1/2-in. strand is 0.519 lb/ft, and the planned weight of epoxy coating was 0.032 lb/ft. The actual average coating weight was 0.035 lb/ft. Coating thickness was measured all over all six exterior wires and also between the six wires in the valley section of the strand. The measured data show the following:

- Average thickness over the six exterior wires ranged from 32 to 40 mils and averaged 36.1 mils.
- Actual minimum thickness over any individual exterior wire was 22 mils.
- Actual maximum thickness over any individual exterior wire was 63 mils.
- Average thickness over the valleys between the six exterior wires ranged from 70 to 84 mils and averaged 76.7 mils.
- The coefficient of variation on the coating thickness over the six exterior wires was relatively constant at 20 to 30 percent.

The uncoated bare prestressing strands for System 7 utilized commercially produced strands which were not cleaned with solvent. During the wire drawing process the wires are coated with zinc phosphate and calcium stearate materials, which impart an unintentional but potentially effective corrosion protection to the assembled 7-wire strand. It was decided to use as-produced strand (not cleaned) for System 7 and add a

System 7A test series which would use ultrasonically-cleaned strand for comparative purposes.

The ASTM A767 galvanized bars(26) were chromate-dipped at the factory instead of adding a chromate admixture into the concrete. This decision was based upon the favorable economics of factory dipping versus having each ready mix or precast plant add an additional admixture to the concrete to eliminate hydrogen gas evolution problems. A study was conducted to determine the minimum, maximum, and typical average thickness of the zinc (Zn) coating on the No. 4 galvanized rebars. The bars were cut and subsequently ground with alumina paper to reveal the interface between the steel and zinc. The samples were then examined under a stereo microscope. The minimum and maximum zinc thicknesses were estimated. In addition, the range of zinc thickness representing about 90 percent of the rebar surface was estimated. The results are reported below.

Range of minimum thickness (mils)	Range of maximum thickness (mils)	Typical thickness (mils)
3 to 5	10 to 17	5 to 8

The above typical zinc thicknesses are in accordance with the ASTM A767 requirements for Class I coating weight for No. 4 bars which are 1068 grams/m^2 or about 0.006 in. of thickness.

Concrete Details

The following materials were used:

Sand:	Natural river sand (chloride free)
Coarse aggregate:	3/4-in. maximum size natural, rounded, granitic river gravel (chloride free)
Portland cement:	Type I
Air-entraining agent:	Neutralized vinsol resin

Ten trial batches of air-entrained concrete were initially produced to evaluate w/c ratio, slump, air content, and yield for the conventional concretes and the concretes containing calcium nitrite. The average actual w/c ratios, etc., are tabulated below for the numerous batches of conventional concrete utilized in the production of these slab specimens.

Target w/c	0.50	0.40	0.32
Actual w/c	0.51	0.40	0.28
Cement factor (bags/cu yd)	4.60	6.08	7.47
Unit weight (pcf)	145.9	146.2	147.1
Slump (in.)	3.25	3.0	3.25
Net air (%)	5.9	5.7	6.0

The concrete mixtures for System 6 contained 4 gals/cu yd of calcium nitrite solution. The calcium nitrite solution has a specific gravity of about 1.28. The solids content was stated to be 30 percent. The nitrite ion (NO^-_2) content of each gallon therefore would be 2.22 lbs. Thus, the nitrite ion added per cu yd of concrete was 8.88 lbs. The mixes used for the System 6 specimens had the following average properties.

Target w/c	0.50	0.40	0.32
Actual w/c	0.53	0.44	0.34
Cement factor (bags/cu yd)	4.60	6.03	7.51
Unit weight (pcf)	146.4	145.8	148.2
Slump (in.)	2.0	3.5	2.25
Net air (%)	5.8	5.4	4.1

The cement contents required to achieve the target w/c ratios of 0.50, 0.40 and 0.32 were 4.6, 6.0 and 7.5 bags/cu yd, respectively. The average 28-day strengths for the conventional concretes with 0.51, 0.40 and 0.28 w/c ratios were approximately 5000, 6000 and 7400 psi, respectively. The average 28-day strengths for the concretes with calcium nitrite with 0.53, 0.44 and 0.34 w/c ratios were approximately 5600, 5600 and 7800 psi, respectively.

Specimen Fabrication Details

The 18-in. long normal gray reinforcing bars and prestressing strands were coated with epoxy resin to provide corrosion protection for the 3-in. lengths that protruded from the ends of the slabs. Three coats of the type of epoxy normally used to repair holidays in fusion bonded epoxy-coated bars were applied to a nominal 3 1/2-in. length at each end. Thus, only 11 in. of the uncoated bar or strand was in contact with the concrete within each slab.

The reinforcing bars or strands were positioned in a wooden form. Neither the bars nor the strands were in electrical contact with each other at this stage. The slabs were cast with the two potentially anodic bars at the bottom of the form during the casting operation. Following normal ASTM quality control tests for slump, air content, and unit weight, the concrete was placed and table-vibrated to provide consistent consolidation. The top surface as cast became the bottom surface or potentially cathodic region during the cyclic testing.

The slabs were moist cured overnight under wet burlap and plastic sheeting. At age one day, the slabs were stripped and put into plastic bags for two additional days of curing. They were then allowed to air dry in the laboratory air from age three days.

During the air drying period, the macrocell circuitry instrumentation was installed, the 4 sides were coated with 2 coats of a 50 percent solids epoxy coating, the dikes were installed and the protruding bars were again coated with epoxy resin after all the electrical systems were installed.

Chloride Ion Content Details

The drilled powder was tested for acid-soluble chloride ion content using a test procedure which produces equivalent test results to those obtained by the AASHTO T 260-82 test method.(27)

Sealing and Coating Details for Systems 9 and 10

These specimens were provided 3 days of moist curing as previously described and then allowed to air dry for 18 days in the 60 to 80°F laboratory air. The coating materials were applied to the as-cast surfaces at age 21 days using the following application rates:

System 9 - One coat of 40 percent solids alkyl-alkoxy silane in alcohol at 125 sq ft/gal

System 10 - One coat of oligomeric alkoxy silane primer at 100 sq ft/gal, followed the next day with one coat of methyl methacrylate finish coat at 200 sq ft/gal

Both materials for Systems 9 and 10 were previously used in the project described in the NCHRP 244 report(17) using similar or the same coverage rates. They were identified as materials No. 6 and 8, respectively, in that previous research study.

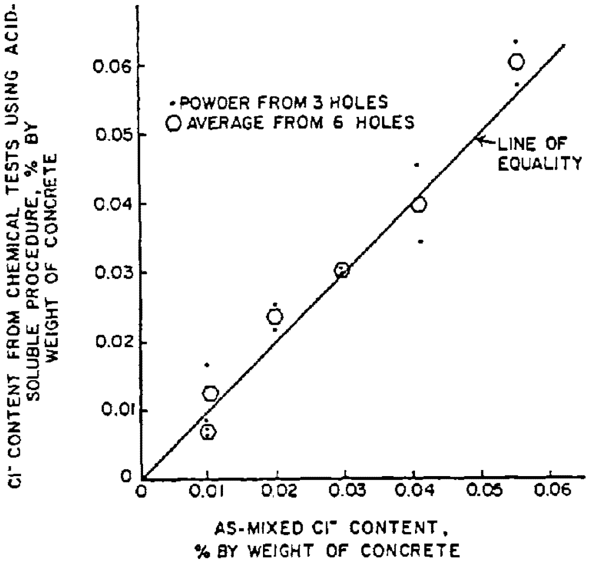
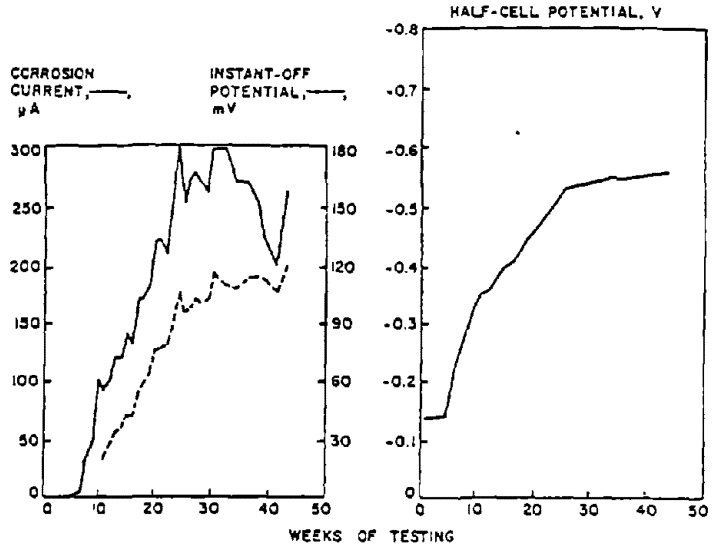


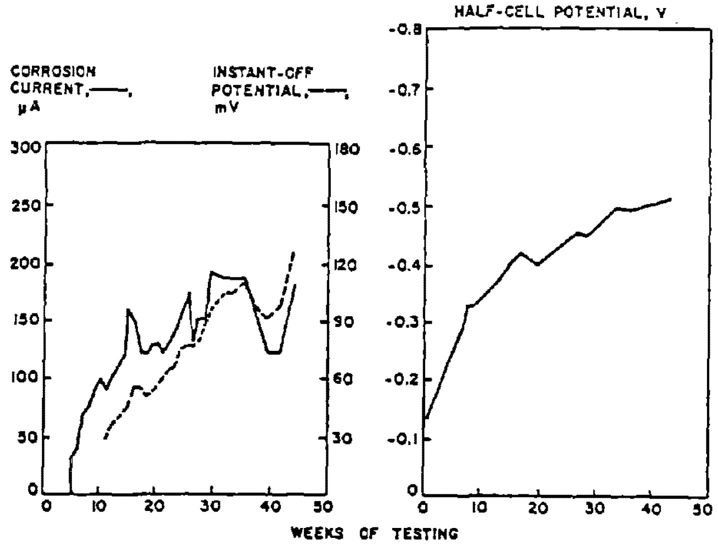
Fig. 31 - Relationship between "as-mixed" and "as-tested" chloride ion contents by analyzing powder from 1/4-in. diameter drilled holes from pilot time-to-corrosion study.

APPENDIX C

MEASURED CORROSION ACTIVITY DATA FOR PILOT STUDY SPECIMENS EXHIBITING CORROSION AND FINAL CHLORIDE CONTENT DATA

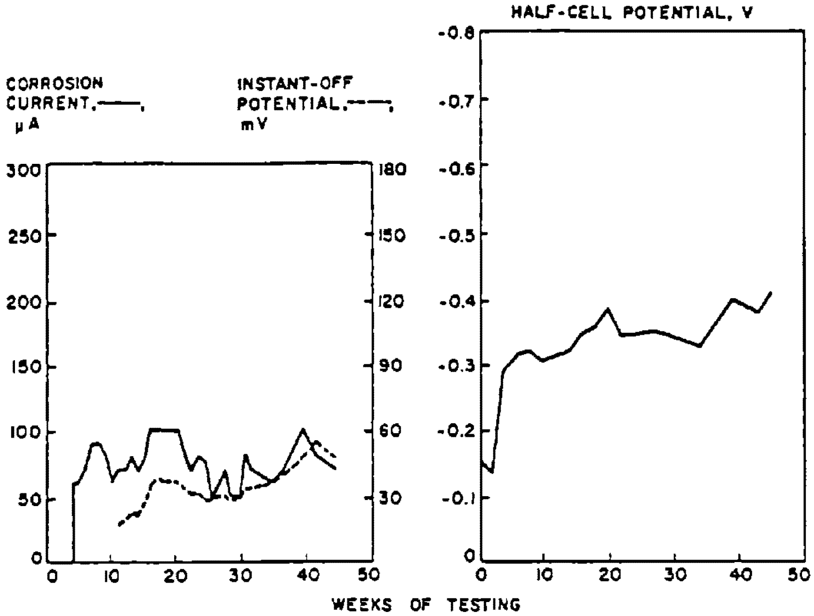


SPECIMEN NO. A

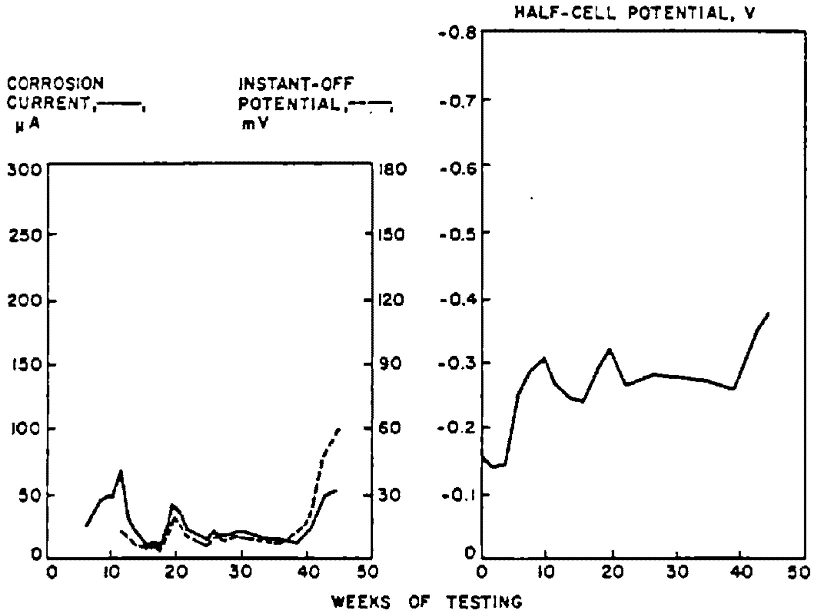


SPECIMEN NO. B

Fig. 32 - System 1 Corrosion Activity Data, 1-in. cover and 0.50 w/c.

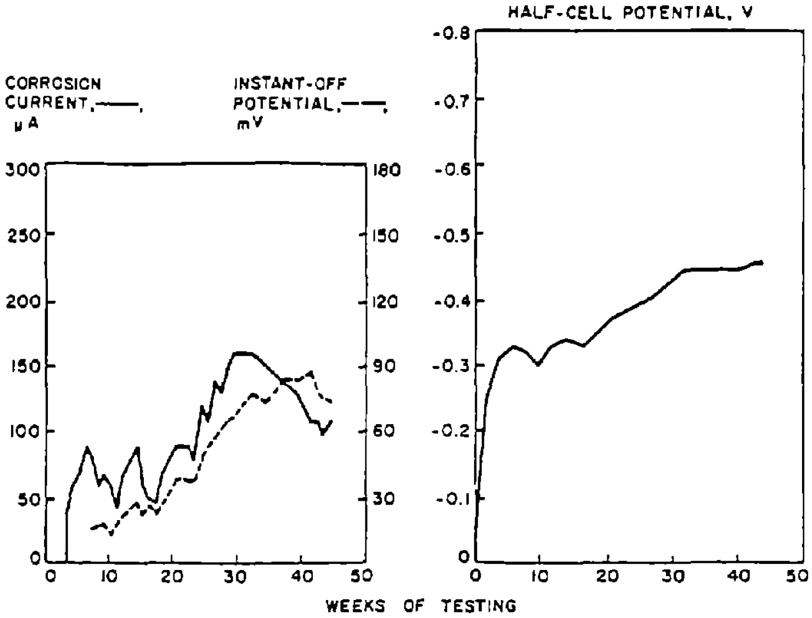


SPECIMEN NO. A

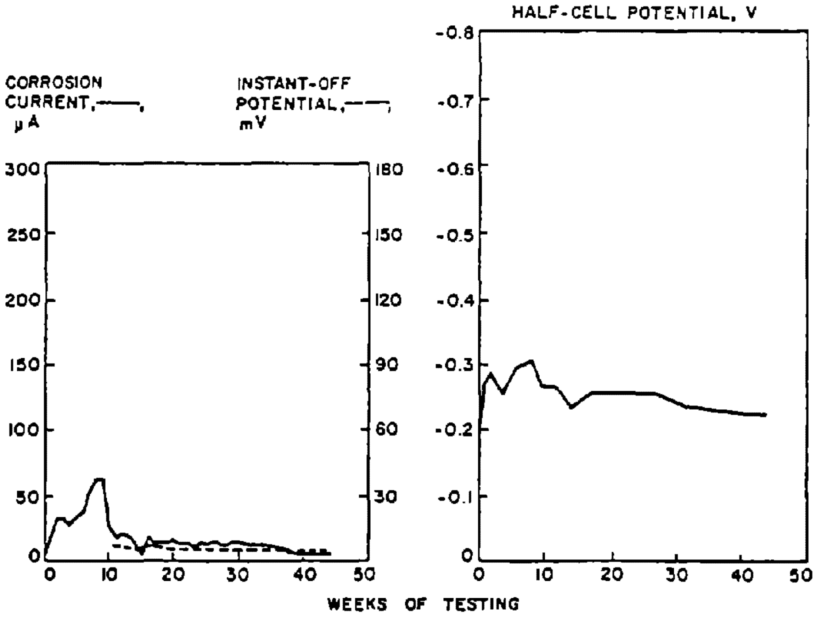


SPECIMEN NO. B

Fig. 33 - System 1 Corrosion Activity Data, 1-in. cover and 0.40 w/c.

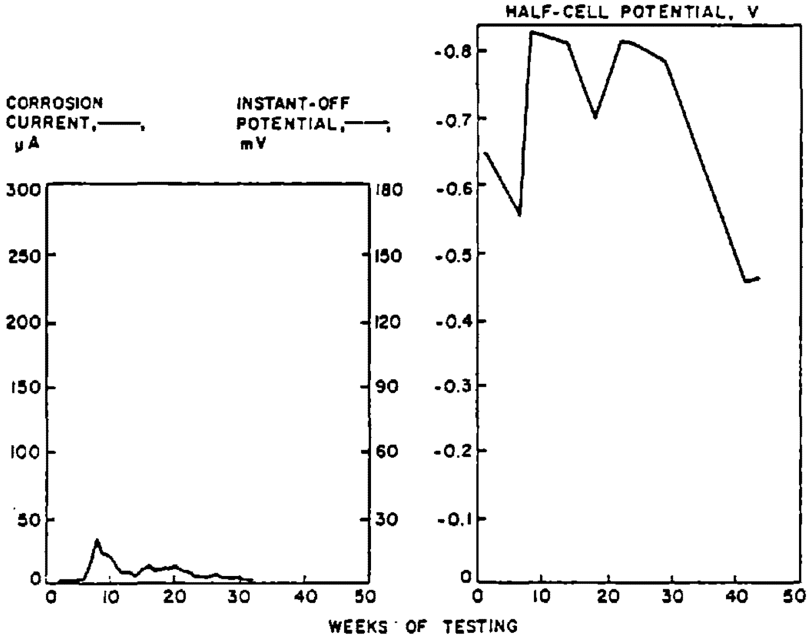


SPECIMEN NO. A

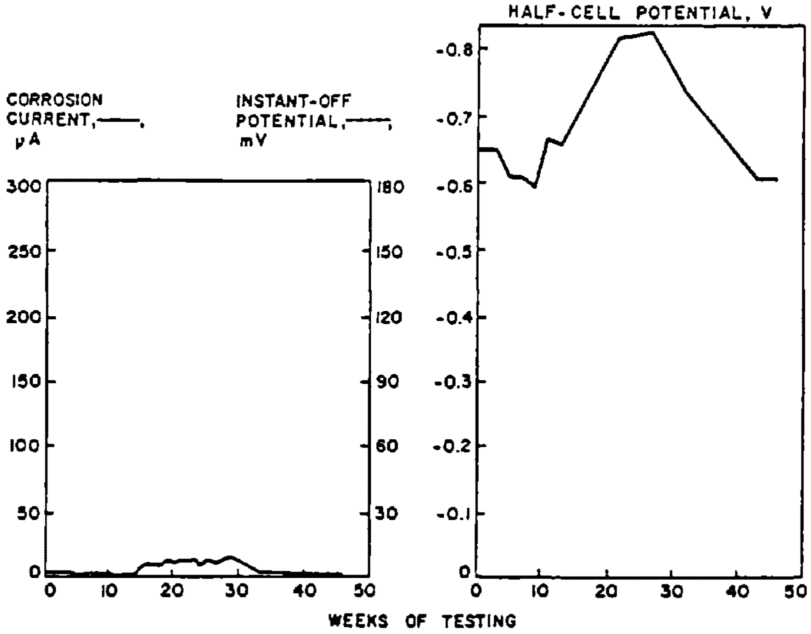


SPECIMEN NO. B

Fig. 34 - System 1 Corrosion Activity Data, 1-in. cover and 0.32 w/c.

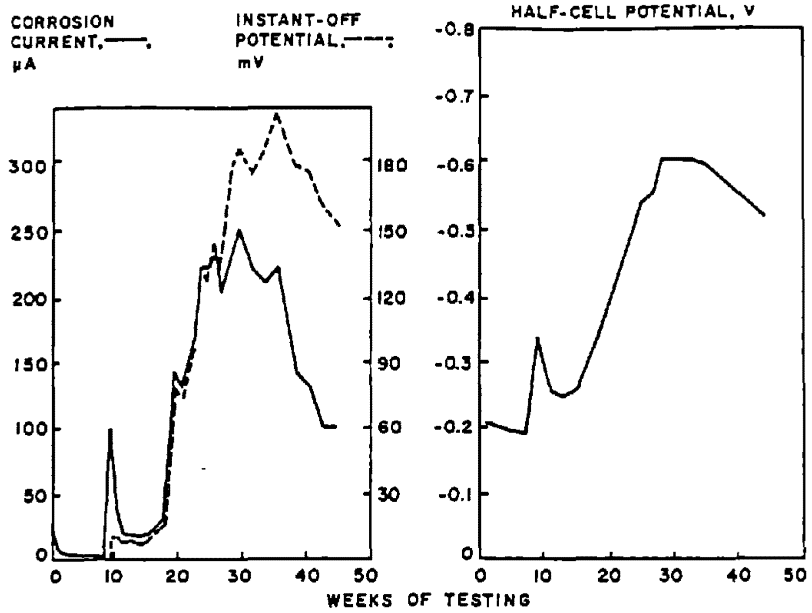


SPECIMEN NO. A

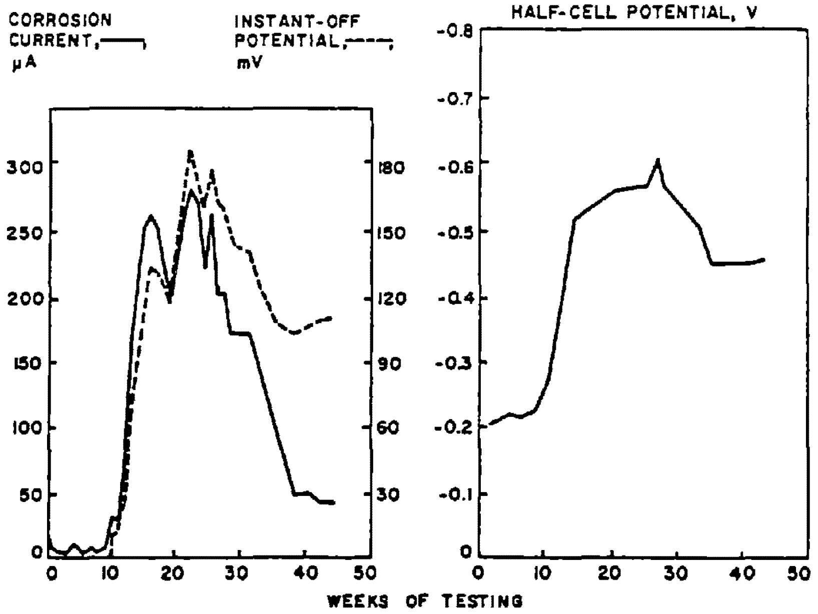


SPECIMEN NO. B

Fig. 35 - System 4 Corrosion Activity Data, 1-in. cover and 0.50 w/c.

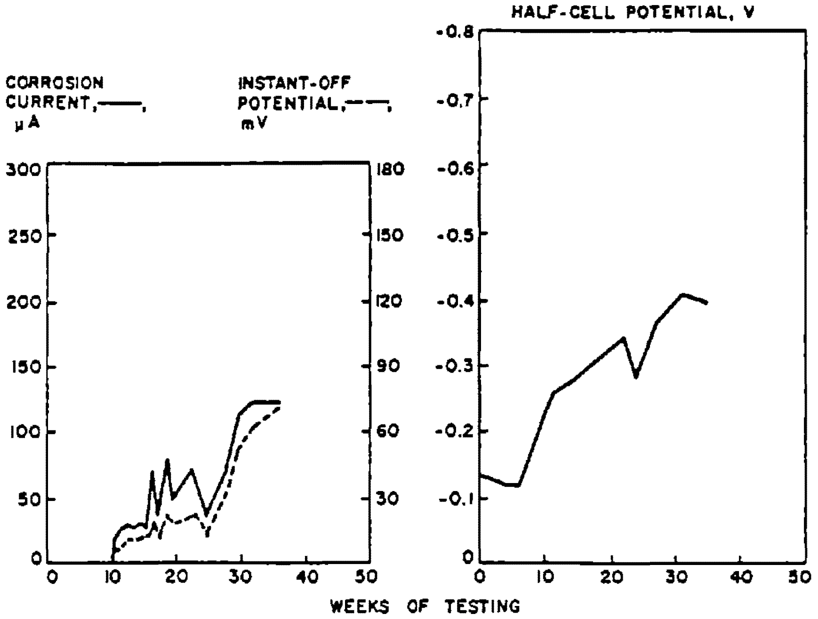


SPECIMEN NO. A

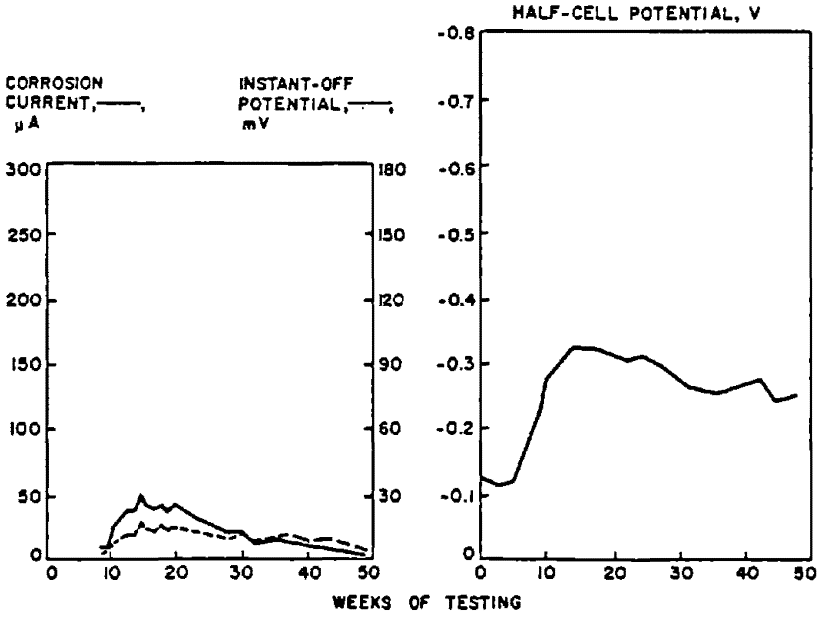


SPECIMEN NO. B

Fig. 36 - System 5 Corrosion Activity Data, 1-in. cover and 0.50 w/c.

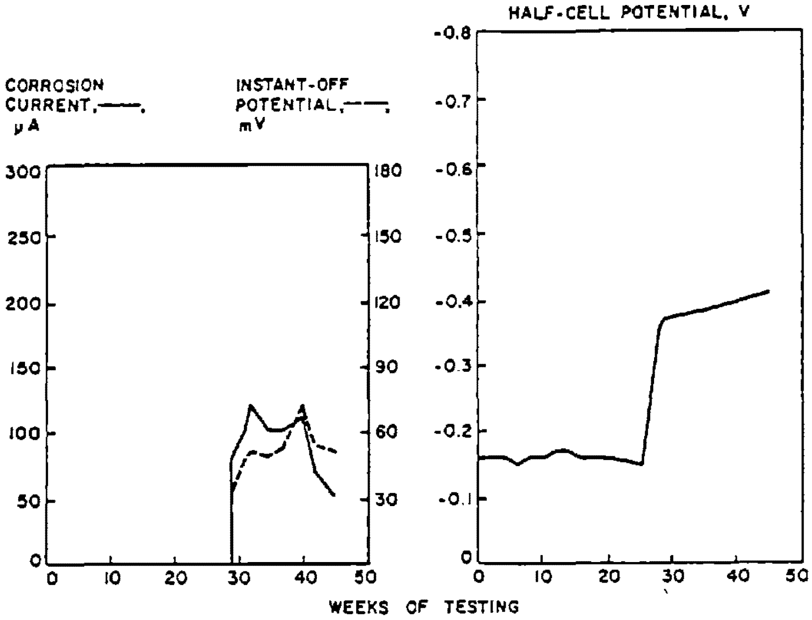


SPECIMEN NO. A

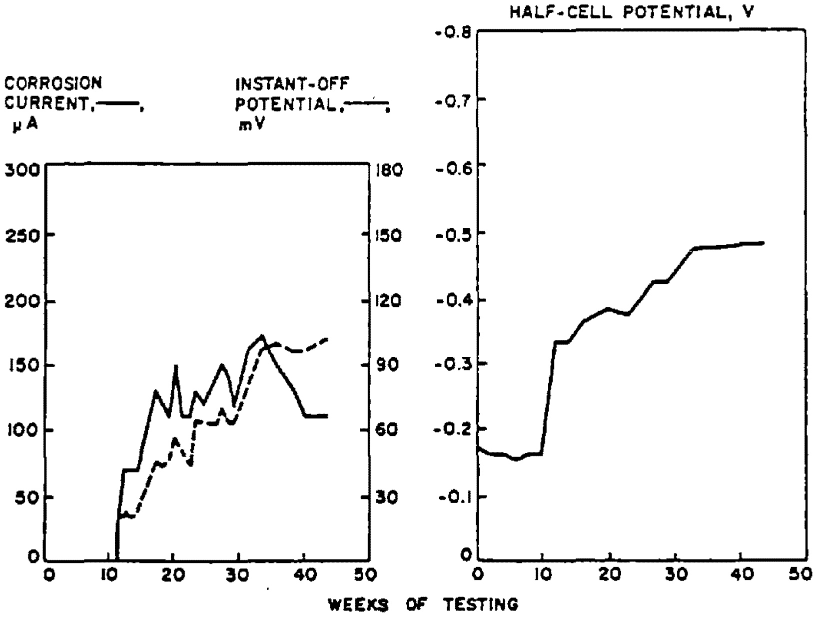


SPECIMEN NO. B

Fig. 37 - System 6 Corrosion Activity Data, 1-in. cover and 0.50 w/c.

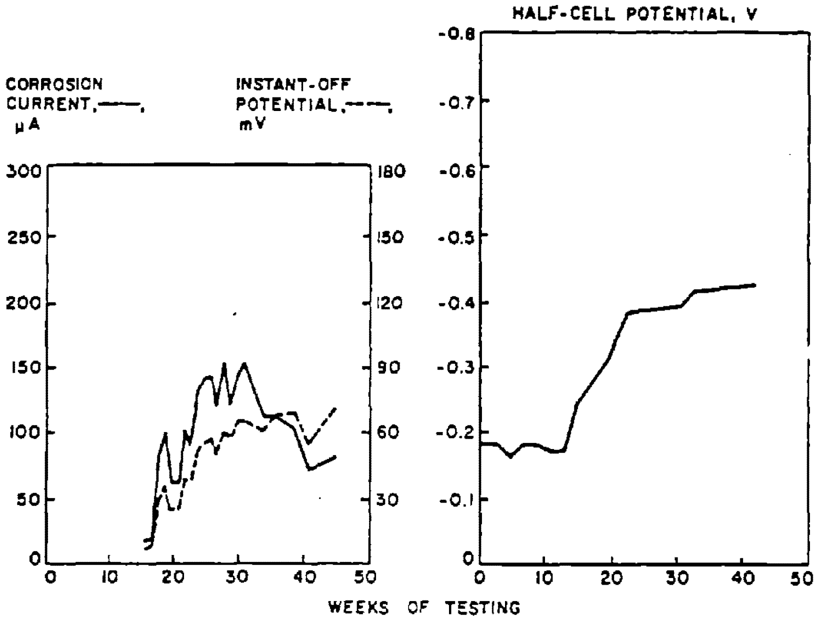


SPECIMEN NO. A

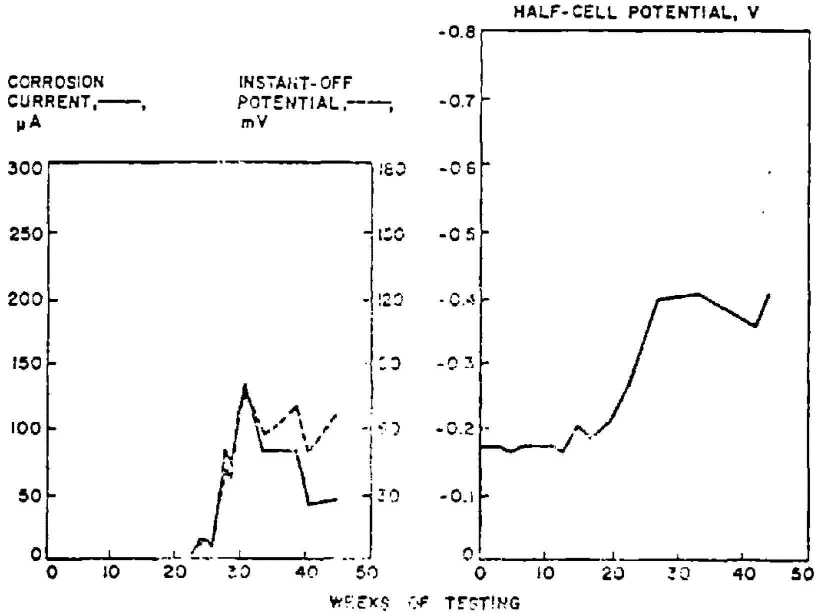


SPECIMEN NO. B

Fig. 38 - System 7 Corrosion Activity Data, 1-in. cover and 0.50 w/c.

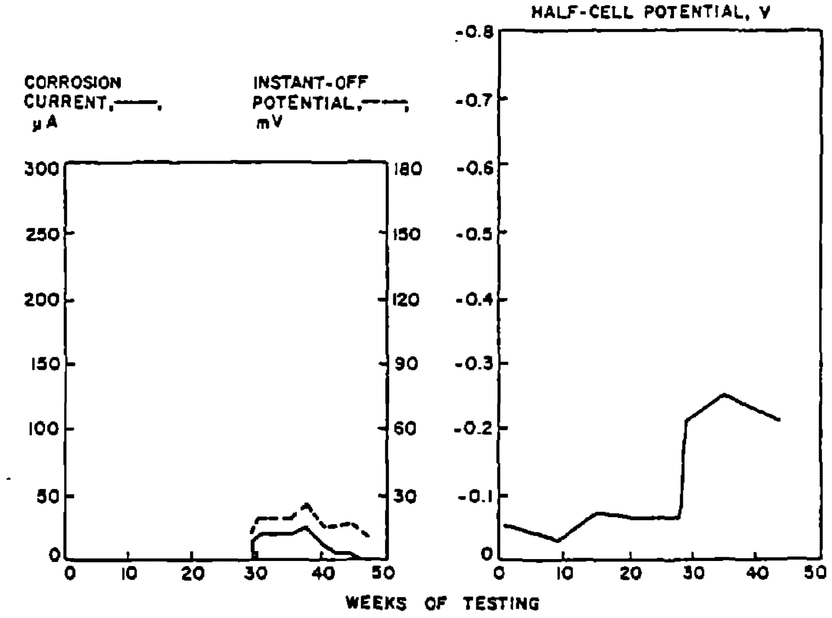


SPECIMEN NO. A



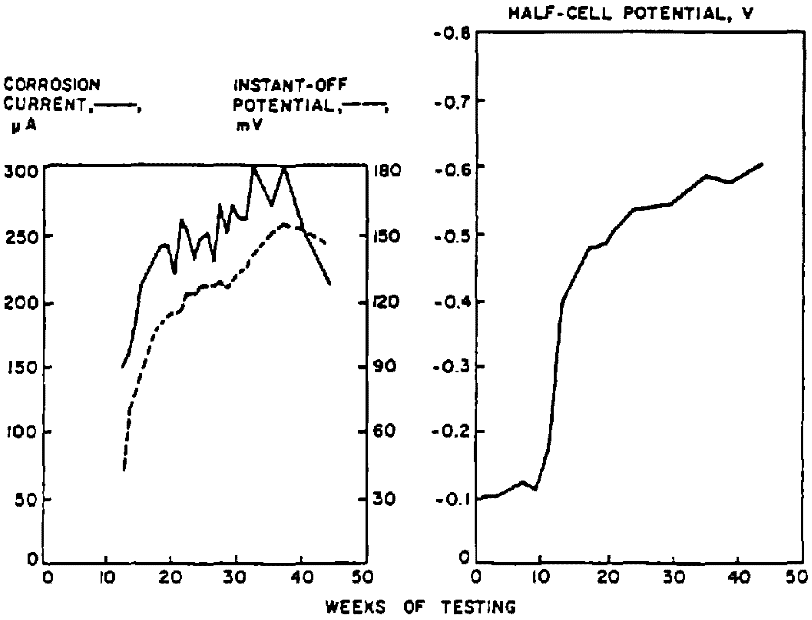
SPECIMEN NO. B

Fig. 39 - System 7A Corrosion Activity Data, 1-in. cover and 0.50 w/c.

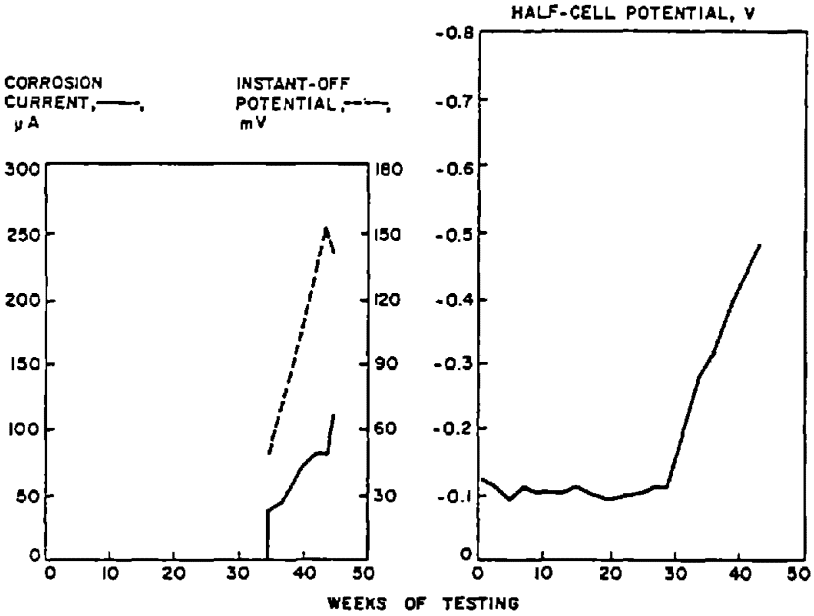


SPECIMEN NO. A

Fig. 40 - System 9 Corrosion Activity Data, 1-in. cover and 0.50 w/c.

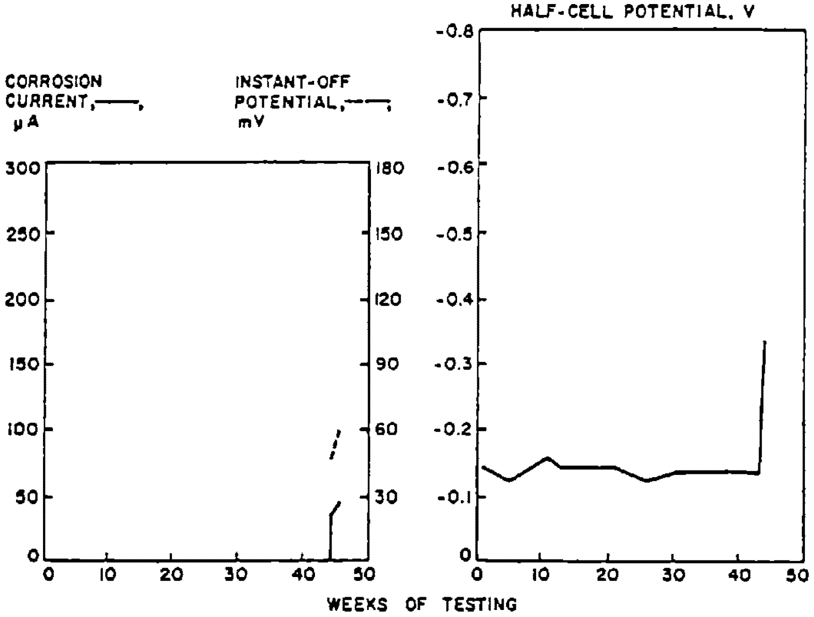


SPECIMEN NO. A



SPECIMEN NO. B

Fig. 41 - System 10 Corrosion Activity Data, 1-in. cover and 0.50 w/c.



SPECIMEN NOS, A AND B

Fig. 42 - System 10 Corrosion Activity Data, 2-in. cover and 0.50 w/c.

Table 8 - Final chloride contents from pilot time-to-corrosion slabs.

System No.	Sample depth (in.)	Nominal w/c ratio	Cl^- content at 44 weeks, (% by wt of concrete)	
			Slab A	Slab B
1	1	0.50	0.443	0.407
	2	0.50	0.019	0.014
	3	0.50	0.018	0.029
1	1	0.40	0.079	0.046
	2	0.40	<0.004	<0.004
	3	0.40	<0.004	<0.004
1	1	0.32	0.030	0.027
	2	0.32	0.006	0.025
	3	0.32	0.009	0.007
2	1	0.50	0.429	0.493
	2	0.50	0.030	0.011
	3	0.50	0.008	<0.004
2	1	0.40	0.165	0.099
	2	0.40	<0.004	<0.004
	3	0.40	<0.004	<0.005
2	1	0.32	0.032	0.011
	2	0.32	0.017	0.018
	3	0.32	0.005	0.005
3	1	0.50	0.349	0.420
	2	0.50	0.006	0.011
	3	0.50	<0.004	0.004
3	2	0.40	<0.004	<0.004
3	2	0.32	0.005	<0.007
4	1	0.50	0.507	0.504
	2	0.50	0.011	0.012
	3	0.50	0.011	0.011
4	2	0.40	0.015	0.007
4	2	0.32	0.004	<0.004
5	1	0.50	0.396	0.582
	2	0.50	0.030	0.017
	3	0.50	0.017	<0.007
5	2	0.40	0.010	<0.007
5	2	0.32	<0.007	<0.007
6	1	0.50	—	0.252
	2	0.50	—	0.008
	3	0.50	—	0.006
6	2	0.40	—	0.010
6	2	0.32	—	0.006
7	1	0.50	0.498	0.488
	2	0.50	0.014	0.011
	3	0.50	0.004	0.006
7	2	0.40	<0.004	<0.004
7	2	0.32	0.009	<0.007
7A	1	0.50	0.482	0.371
	2	0.50	0.012	0.012
8	1	0.50	0.444	0.405
	2	0.50	0.051	0.048
	3	0.50	<0.007	0.018
8	2	0.40	0.012	<0.007
8	2	0.32	0.015	0.027
9	1	0.50	0.021	0.021
	2	0.50	0.008	0.009
	3	0.50	0.007	0.009
9	2	0.40	0.008	0.016
9	2	0.32	0.007	<0.007
10	1	0.50	0.670	0.219
	2	0.50	0.009	0.022
	3	0.50	0.016	0.017
10	2	0.40	0.012	0.009
10	2	0.32	<0.007	<0.007

APPENDIX D

DETAILS OF REINFORCING STEELS, CONCRETE MIXTURES, SPECIMEN FABRICATION, CHLORIDE ION TESTS, AND COATING PROCEDURES FOR FULL-SIZE SPECIMENS

Reinforcing Steel Details

The No. 4 and 6 reinforcing bars and the 1/2-in., 270-ksi prestressing strands were obtained from commercial sources in Illinois, Maryland, Florida and Indiana. Suppliers stipulated that all normal bars, fusion-bonded epoxy-coated bars, normal strands, fusion-bonded epoxy-coated strands, and galvanized bars met their respective ASTM specifications. The ASTM A767 galvanized bars were chromate-dipped at the factory.

The fusion bonded epoxy-coated No. 6 bars were tested for coating thickness at 234 locations on twelve 6-ft long bars and on twelve 4-ft long bars. The average coating thickness was 8.1 mils with a standard deviation of 1.4 mils. These same 24 bars were also tested for holidays. The average number of holidays per foot was 0.66 for both bar lengths.

Concrete Details

The following materials were used:

Sand:	Natural river sand (chloride free).
Coarse aggregate:	3/4-in. maximum size natural, rounded, granitic river gravel (chloride free).
Portland cement:	Type I.
Air-entraining agent:	Neutralized vinsol resin.
Silica fume admixture:	A dry silica fume concrete admixture that meets the ASTM C494-82 standard specifications as a Type F chemical admixture (high-range water reducer) and ASTM C618-78 as a pozzolanic mineral admixture.

Calcium nitrite admixture	A corrosion-inhibiting chemical admixture which contains 30 percent calcium nitrite, and approximately 67 percent water. The specific gravity of the solution is 1.28.

Trial concrete mixes were initially made for the three different concretes. Mixtures were produced with and without calcium nitrite at the nominal 0.44 w/c ratio. The nominal cement content for both of these mixtures was established at 6 bags/cu yd. These two concretes contained 6 ± 1/2 percent air. The slump was 4 ± 1 in. Calcium nitrite was used at a dosage of 5.4 gallons/cu yd, which is equivalent to 3.1 percent calcium nitrite by weight of cement or 12.1 lbs of nitrite ion/cu yd. This dosage was 35 percent greater than the 4.0 gal/cu yd used in the Pilot Study.

The concrete mix design containing the silica fume admixture had a nominal cement content of 8.5 bags/cu yd. Silica fume was added at the rate of 20 percent powder by weight of cement. The resultant w/c ratio was 0.22 while the water/cement plus silica fume ratio was 0.18. This concrete was also air entrained at 6 ± 1/2 percent, and the slump was 6 ± 2 in. The AEA content demand for the silica fume concrete was 36.6 oz/cu yd or 8 1/2 times that required to entrain the same amount of air in the normal concrete or the calcium nitrite concrete.

Mix proportions for the full-size specimens as based upon the trial mixes are shown in Table 9.

The overnight heat-cured compressive strengths of the two 0.44 w/c ratio concretes and the silica fume concrete were about 3000 to 3500 psi and 7000 psi, respectively. The 28-day compressive strength for the moist-cured and heat-cured 0.44 w/c ratio concretes (with or without calcium nitrite) ranged from 5400 to 6100 psi and averaged 5840 psi. The average 28-day compressive strength for the silica fume concrete was 9,500 to 10,500 psi.

Table 9 - Full-size specimen mix designs.

Material	SSD quantities, per cu yd Conventional concrete mix	 Calcium nitrite admixture mix	 Silica fume admixture mix
Type I cement (lbs)	560	560	800
Eau Claire sand (lbs)	1230	1230	1060
Eau Claire coarse aggregate (lbs)	1850	1850	1750
Vinsol resin AEA (oz)	4.3	4.3	36.6
Calcium nitrite (gal)	--	5.4	--
Silica fume admixture (lbs)	--	--	160
Water (lbs)	247	247*	175

*Includes water from calcium nitrite admixture.

Typical Specimen Fabrication Details

The reinforced concrete columns and beams contained twelve No. 6 bars (four in each face) with 1-, 1 1/2- or 2-in. clear cover. These eleven specimens were cast in wooden forms with their lengths vertical. The twelve bars were not in electrical contact with each other and column ties or stirrups were not used. Following the normal ASTM quality control tests for slump, air content and unit weight, the concrete was placed and internally vibrated. The columns and beams were moist cured overnight under wet burlap and plastic sheeting. At the age of one day, they were stripped and covered with wet burlap and plastic sheeting for two additional days. They were then uncovered and allowed to air dry in the laboratory at 60 to 80°F from the age of three days. During this air-drying period, the corrosion circuitry and the saltwater circulating systems were installed. The cyclic tests began on these specimens, at about an age of 50 days.

The pretensioned, prestressed concrete piles and stay-in-place bridge deck panels were cast and cured using typical precasting production and heat-curing methods. A 10-ft long, 300-kip prestressing bed was fabricated. The 16 in. square solid pile was symmetrically prestressed with eight 1/2-in. (270 ksi) strands. These eight strands were not in electrical contact with each other, and stirrups or lateral ties were not used. The piles were cured with a typical overnight accelerated radiant heat-curing system. Following casting, the specimens were covered with wet burlap and plastic film. The concrete was then allowed to achieve initial set, as determined by ASTM C403. At the time of initial set, about 4 hrs, the radiant heat system was started. The air temperature increased about 20°F per hr to a maximum air temperature of about 130° ± 10°F. This maximum air temperature was maintained until detensioning time the next morning.

Each strand was tensioned separately. The prestressing force was monitored with load cells during tensioning and the overnight heat curing. The average stress in the strands immediately after tensioning was 185,600 psi, which is 69 percent of the ultimate strength. The average stress in the strand after overnight heat curing at 130°F just prior to detensioning was 175,000 psi. Thus, the stress in the concrete in the 16 in. square piles at detensioning was equal to a uniform compression of 840 psi. Following detensioning, the piles were removed from the wooden form and allowed to air dry at 60 to 80°F laboratory conditions with no supplemental moist curing. The cyclic tests began at about the age of 35 days.

The pretensioned, prestressed concrete bridge deck panels were 2 1/2-in. thick and 24-in. wide in the cyclic test region. This precast deck panel was symmetrically prestressed with two 1/2-in. (270 ksi) strands. One conventional No. 4 gray bar was also located at the same mid-depth location for comparison of the corrosion behavior of prestressed strands and reinforcing bars. These three pieces of reinforcement in the deck panel were not in electrical contact, and transverse reinforcing bars were not used in the deck panel. Following the casting operation, the top surface of the deck panel was deeply raked to produce 1/4-in. high undulations to help achieve good bond to the subsequently cast 5 1/2-in. thick composite reinforced concrete wearing course layer. The overnight heat-curing procedure was the same as that used for the piles. The average uniform stress in the concrete in the deck panel immediately after overnight heat curing and detensioning was 900 psi in compression. Following detensioning, the deck panels were removed from the wooden forms and allowed to air dry with no supplemental moist curing. At age 15 to 20 days, the precast deck panels were positioned horizontally and the 5 1/2-in. thick conventionally-reinforced concrete wearing course layer was cast onto the deck panels. This layer was reinforced with longitudinal and transverse No. 6 gray bars which were in electrical contact to each other. Normal ready-mixed concrete approximating the quality of the laboratory-produced concrete used in the subdeck panel was used. This 5 1/2-in. layer was moist cured under wet burlap and plastic for 3 days. The cyclic tests began at about age 40 days.

Chloride Ion Content Tests

Chloride ion content tests were made at the conclusion of the 370-day cyclic testing. Concrete cores with a net diameter of 1 3/4 or 2 1/4 in. were drilled at various locations to a depth of 4 in. Each core was then cut into 3/8 in. thick slices at depths from the saltwater exposed surfaces of 1/2, 1, 1 3/4, 2 1/2 and 3 1/4 in. The 3/8-in. thick slice was centered on the above depth increments. Each slice was then crushed to powder and tested for acid-soluble chloride ion content using a test procedure which produces equivalent test results to those obtained by the AASHTO T 260-82 test method.

Tests to determine the chloride ion content at time-to-corrosion were not made during these full-size member tests.

Since these various bridge members were exposed to horizontal and vertical water flow as well as cyclic and constant saltwater exposure, a number of cores were taken from each specimen to establish average chloride penetration as outlined below:

Specimen Type	Number of cores per specimen	
	Horizontal water flow	Vertical water flow
Columns	--	4
Beams	2	2
Subdeck bridge panels	--	2
Piles:		
ponded region	2	--
wet/dry region	2	--
air-dry region	1*	--

*Only one pile tested

The location for these various cores are described below:

Horizontal water flow

Specimen type	Number of cores per specimen	Location of cores
Beams	2	Near centerline of beam on top horizontal surface
Piles	1*	Near centerline of pile on top surface in constantly air-dry region
	2	Near centerline of pile on top surface in wet/dry region
	2	Near centerline of pile on top surface in constantly ponded region

*Only one pile tested in air-dry region.

The above schedule required a total of 10 cores from the 5 beams and 17 cores from the 4 piles for the horizontal water flow chloride ion distribution study.

Vertical water flow

Specimen type	Number of cores per specimen	Location of cores
Columns	2	Top region, 9 in.below water discharge pipe
Columns	2	Bottom region, 51 in. below water discharge pipe (9 in. above bottom of column)
Beams	2	9 in. below horizontal edge
Prestressed sub-deck bridge panel	2	7 in. below water discharge pipe

The above schedule required a total of 24 cores from the 6 column specimens, 10 cores from the 5 beams and 8 cores from the 4 bridge deck panel specimens for the vertical water flow chloride ion distribution study.

A grand total of 69 cores were taken and 345 chloride ion contents were measured.

Silane Treatment Details

The specimens C3, B5 and BD3 were each treated with a silane penetrating sealer prior to cyclic testing at the age of about 20 days. The normally air-dried test surfaces were sealed with 1 coat of 40 percent concentration alkyl-alkoxy silane in alcohol at 125 ft/gal. The silane solution was applied by paint brushes.