

II: Fundamental Factors Causing Corrosion

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Available construction records show that, judged by present day standards, the water-cement ratio of the concrete in the San Mateo-Hayward Bridge was high for the conditions of exposure. It is evident that the concrete has suffered from sulfate attack to a moderate, but not yet critical, degree. The concrete has relatively high absorption and permeability. However, the concrete shows little outward indication of distress other than severe cracking in the plane of main reinforcing members, accompanied by severe spalling at some locations.

It is evident that rupture and spalling are primarily the result of pressure caused by corrosion products of the reinforcing steel which have characteristics differing from those found on steel subjected to atmospheric corrosion. No evidence could be found that stray currents had produced electrolysis.

Chlorides in samples of the concrete indicate that substantial quantities of sea salts have been absorbed. Characteristically the concentration of salt varies from point to point and for this reason macro galvanic corrosion cells of the differential concentration type have been established. Electrical potentials approaching 0.5 have been measured between points 2 to 10 ft apart. Potential measurements over a systematic grid on the surface of members indicate the existence of numerous, distinct anodic and cathodic areas. The rate of corrosion is determined by the activity of the cell. Variations in average atmospheric humidity at different parts of the bridge are believed to explain variations in the rate of corrosion.

When affected concrete is removed and replaced with new shotcrete, the pattern of cathodic and anodic areas is changed.

Laboratory tests have confirmed the role of salts and moisture in promoting corrosion. Both laboratory and field studies are being continued.

Work on two experimental methods of eliminating or retarding future corrosion is under way at the bridge site. This consists of (a) cathodic protection and (b) the application of a coating impervious to oxygen to the surface of cathodic areas of the deck units.

●PART I has described the San Mateo-Hayward Bridge and has given details of design, methods of construction, and service history. Emphasis was placed on the occurrence of longitudinal cracks in the plane of the main reinforcing steel. These began to develop during the early life of the bridge and have continued to develop at an accelerated rate. Steel in the region of the crack was found to be badly corroded. Frequently the corrosion was found to extend beyond the limits of the crack.

Shortly after acquisition of the bridge by the state, the Materials and Research Department of the California Division of Highways was authorized to make an investigation of the basic causes of distress and, if its findings warranted, to make recommendations as to improved methods of repair and maintenance. Part II of the paper describes the investigations that have been made to date.

NATURE OF THE CORROSION

One of the first observations made by the investigators was that the reinforcing steel is deeply pitted, whereas in steel corroded under direct exposure to marine atmospheres extreme pitting is uncommon. Where corrosion has occurred within concrete that has not cracked the products are dark grey to brown in color and tend to be moist

and semi-solid. Where the concrete has been ruptured, the deposits generally resemble those found under direct exposure to marine atmosphere except that they are accompanied by pitting of the reinforcement. This finding is indirect evidence that the corrosion was the cause, and not the result, of the longitudinal cracks in the concrete. It points to the conclusion that corrosion was produced by large-scale electro-chemical reactions within the intact body of the concrete.

It has been reported by others (7, 10, 11, 12, 13, 15, 16, 17, 18, 19, 20, 22) that corrosion of reinforcement has been caused by stray electrical currents of sufficient potential to cause corrosion of steel within the concrete. The action has been produced in the laboratory by impressed currents. It has been found that the corrosion products occupy 2.2 times as much space as the metal and may develop mechanical pressure as high as 4,700 psi, a force many times greater than the tensile strength of concrete (13).

Investigations were made to explore the possibility that electrolysis by stray currents may have been responsible for the rupture of concrete in the San Mateo-Hayward Bridge. Tests for stray electrical currents were made on the ground at the west end of the bridge and on the surface water near the center of the bridge. No potential gradient to or from the bridge was detected. It was established that the electric system on the bridge had never been direct current. Adjacent spans of the bridge were not designed with interconnected reinforcement and tests established that they were not in fact electrically connected. These findings indicate quite conclusively that stray currents were not and could not be present in the bridge as a whole. The possibility that corrosion was caused by electrolysis appears to be definitely ruled out.

Inasmuch as neither atmospheric action nor stray electrical currents were responsible for the corrosion, it must have been produced by galvanic cells within the concrete. Test results show that these are macro-cells in which anodes and cathodes are separated by distances measurable in feet rather than in fractions of an inch.

CORROSION OF STEEL IN AN ELECTROLYTE

Corrosion of buried steel pipelines has been reported extensively in the literature. One of the causes of such corrosion has been established as variations from point to point along the line in the concentration of salts, or other ionizable compounds in the soil, which functions as an electrolyte. Corrosion cells of this type, called differential concentration cells, are believed to be of major importance in the corrosion of the steel in the San Mateo-Hayward Bridge. Differences in concentration within the electrolyte, moist concrete, are due to non-uniform distribution of sea salts that have entered the concrete and to non-uniform distribution of absorbed water. It will be shown that the concrete of the San Mateo-Hayward Bridge has characteristics similar to those found in the soils in which corrosion has been a problem in buried pipelines.

The corrosion of steel in an electrolyte has been established as an electrochemical action (1, 2, 3, 4, 5, 6). Basically it is stated that ionizable compounds must be present in solution in order to cause or support the action of corrosion and that the process is accompanied by a flow of direct-current electricity.

Loss of steel occurs at the anodic area and iron goes into solution in the ferrous state. Hydrogen ions are moved to the cathode and hydrogen collects at this point. Metal is not lost at the cathode. Corrosion may be retarded or stopped by the accumulation of precipitated corrosion products at the anode. It may also be impeded by polarization due to the build-up of hydrogen at the cathode. Chemical reactions which remove the collected hydrogen tend to promote continued activity of the cell. Oxygen, if available at the cathode, is an effective depolarizer. Under some conditions the corrosion products originating at the anode may be precipitated at a significant distance from the surface of the metal and then are unable to seriously impede the operation of the cell.

It has been established in the literature, and in practice, that the corrosion of steel can be detected by measurements of the electrical potential of the metal in an electrolyte. Also, corrosion can be stopped by impressed or galvanic currents operating to counteract the corrosion currents, thereby nullifying the cell. Current practice in the study of steel corrosion employs measurements of the potential of the steel to a

standard reference cell and of the electrical resistivity of the electrolyte (7, 8, 9). The significance of these factors will be amplified further in this paper.

CONCRETE AS A FACTOR IN CORROSION

In the absence of electrolysis due to stray or impressed direct electrical currents, concrete for the most part is known to provide adequate protection against corrosion of the imbedded steel reinforcement. To a great extent the degree of protection depends on the thickness of cover over the steel and covers ranging from 2- to 4-in. thick are generally considered to afford adequate protection. There are however, reports in the literature (2, 5, 6, 13, 14, 21, 22, 24) that reinforcement has corroded under covers of this magnitude, and that it has occurred in the absence of stray or impressed electrical currents. These reports make it evident that the inherent inhibiting effect of concrete is seriously reduced by ingress of salts. It has been stated (6, 13) that salt functions in two ways to promote corrosion of steel. The electric conductivity of the salt solution permits the operation of anode and cathode areas farther removed from each other and the corrosion products tend to precipitate at a place removed from the surface of the metal and thus provide a less protective layer.

CHARACTER OF THE BRIDGE CONCRETE

Studies were initiated to determine to what degree the particular concrete used in this structure may have promoted corrosion of the reinforcing steel. Available construction records show that the average cement content of the concrete in the piles is 6.9 sacks per cubic yard and in the deck units, 5.36 sacks per cubic yard. No records pertaining to the cast-in-place caps or diaphragms have been found. Subsequent analyses of samples from the bridge confirm the cement contents as reported and show that the concrete in the caps has a cement factor of 5.4 sacks per cubic yard.

The concrete used in casting the piles and deck units was reported as varying from 3 to 6½ in. in slump, but in the main was within the range of 3 to 5 in. Gravel used as coarse aggregate, nominally of 1¼-in. to No. 4 size, had a high percentage passing 1 in. Construction records pertaining to grading of the sand are meager, but analysis of hardened concrete samples from the bridge show it to be well graded with a fineness modulus of the order of 2.8.

Based on the grading of the aggregate and the slump of the concrete, it can be estimated with reasonable assurance that the water content was about 40 gal. per cu yd. From the cement factors as reported, water-cement ratios are computed to be:

Component	Water, gal./sack
Deck units and caps	7.5
Piles	5.7

The water of San Francisco Bay at the bridge site contains approximately 17,000 ppm of chloride ion, or about 86 percent of the concentration of average sea water. Recommendations of the American Concrete Institute (26) for exposure comparable to that at the bridge site, are water-cement ratios not in excess of 4.5 gal. per sack in piles and 5 gal. per sack in deck units and caps. The water-cement ratios used in the bridge are therefore, from 1 to 2½ gal. per sack higher than present-day good practice would indicate.

Compressive strengths (28-day) of 170 test cylinders made during construction of the piles were in the range of 4,000 to 5,000 psi. Records of 28-day strength of deck units and caps have not been found, but the 7-day strength of three cylinders had an average value of 1,700 psi and this is roughly equivalent to 2,900 psi at 28 days. The average strength of 52 cores cut from the deck units at the age of 27 years was 3,500 psi. This strength is considerably lower than would be expected for concrete of its age subjected to damp conditions and it seems probable that the strength has retrogressed from some previous higher value.

The probability of sulfate attack is indicated by the fact that the cement used reportedly contained 12 percent tricalcium aluminate and that the average content of magnesia now found in the concrete is about 7 percent of the cement compared to 1.7 per-

TABLE 3
ABSORPTION OF 2- BY 4-INCH CORES IN COLD AND BOILING WATER
 (in Percentage by Weight of Oven-Dry Specimens)

	Beams	Caps	Piles	Old Shotcrete	New Shotcrete ^a	
					V	H
Cold water,	6.73	6.79	6.47	2.53		
24 hr	6.29	5.78	6.15	2.51		
	6.47	-	5.55	-		
	6.00	-	-	-		
	6.11	-	-	-		
Average	6.32	6.29	6.06	2.52		
Cold water:						
48 hr, average	6.40	6.38	6.12	2.60		
7 days, average	6.55	6.55	6.25	2.76		
7 days, plus boiling water,						
5 hr, average	7.14	7.48	6.74	2.81	6.53	5.86

^aV = placed vertically; H = placed horizontally.

cent in the original cement. Static modulus of elasticity determined by the secant method up to 2,000-psi compressive stress on cores is of the order of 2,500,000, which is an indication of adverse changes in the concrete.

Notwithstanding these implications, there is little visual evidence of sulfate attack. Pulse velocities measured by the sonoscope at a large number of locations in beams, caps and piles average 12,900, 13,200, and 13,200 ft per sec, respectively. Pulse velocities as low as 10,000 ft per sec have been measured at a few locations. The possibility that sulfate attack may in the future assume serious proportions is evident.

Results of tests for absorption and permeability of 20 2-in. cores drilled from beams, caps, and piles are of interest. These are given in Tables 3, 4, and 5. Oven-dry specimens absorbed from 5.6 to 6.8 percent after 24-hr immersion in water at room temperature. Permeability to air of 1-in. sections was found to be much greater than that of comparable specimens from recent bridge construction. A few specimens, 1-in. thick, permitted passage of small amounts of water under a head of 1/4-in. In a transpiration test in which the lower end of a specimen 3 in. high was immersed in

TABLE 4
PERMEABILITY TO AIR OF 1-INCH SECTIONS SAWED FROM 2-INCH CORES
 (Drop in Pressure^a from 30 psi in 30 min.)

Beams	Caps	Piles	Old Shotcrete	New Shotcrete	Comparison Cores ^b
4	25	6	1	0	1
2	10	1	1	1	1
-	28	6	6	-	1
-	27	18	-	-	-
-	-	4	-	-	-

^aDrop in air pressure in a closed container connected to one face of specimen, the other face being at atmospheric pressure.

^b2-in. cores drilled from column of recent construction.

water and the upper end exposed to the atmosphere, the rate of evaporation of the bridge specimens was relatively high. These tests indicate that the permeability of the bridge concrete is relatively high and thus tends to permit the access of oxygen where it may act as a depolarizer at the cathodes.

Portions of mortar separated from coarse aggregate in cores was placed in sealed flasks containing freshly boiled distilled water with occasional agitation for 7 days. The liquid then had an alkalinity of 1,500 ppm expressed as $\text{Ca}(\text{OH})_2$. Although slight carbonation was indicated in a 1-in. thick section from the outer surface, the tests show that excessive leaching or carbonation of calcium hydroxide had not occurred.

Absorption of sea salt by the concrete was determined as chloride ion in air-dry mortar in the 2-in. cores previously described and in 4-in. cores from beams and roadway slabs of the deck units. In addition, chlorides were determined in fragments broken from areas adjacent to reinforcing steel in corroded and non-corroded condition. The results of these tests are given in Tables 6, 7, and 8. It will be noted that

TABLE 5
RATE OF TRANSPIRATION THROUGH 3-INCH SECTIONS OF 2-INCH CORES
(Grams of Water Evaporated^a During 24-hr Period Preceding the Test Age Shown)

Test Age, days	Beams	Caps	Piles	Shotcrete	New Shotcrete ^b	
					V	H
2	6.0	5.4	1.5	0.0	0.4	2.3
	1.2		0.6		0.3	0.2
	5.0					
	4.2					
	8.7					
	<u>5.0</u>	<u>5.4</u>	<u>1.1</u>	<u>0.0</u>	<u>0.4</u>	<u>1.3</u>
4	4.0	4.0	1.4	0.0	0.2	1.6
	1.4		1.8		0.1	0.3
	3.6					
	3.2					
	5.5					
	<u>3.5</u>	<u>4.0</u>	<u>1.1</u>	<u>0.0</u>	<u>0.2</u>	<u>1.0</u>

^a With lower 1 in. of specimen immersed in water and upper end exposed to evaporative effect of moving air. Apparatus sealed to prevent loss of water except by passage through concrete specimen.

^b V = placing vertically; H = placed horizontally.

salt was more concentrated near the surface of the member and decreased by about one-half at a distance of 2 to 3 in. inward. The average chloride content of the outer inch was 0.10 percent in beams and caps, 0.30 percent in piles, and about 0.04 percent in the lower face of roadway slabs. Values higher than 0.50 percent were found in some fragments from concrete adjoining badly corroded reinforcing steel. Chloride determinations were made on air-dry mortar separated from the coarse aggregate in the samples. Because the mortar comprises about one-half the weight of the concrete and the chloride ion bears a similar relation to total sea salt, the values previously reported are also approximately correct for total sea salt as a percentage of the entire concrete.

The possible concentration of sea salt in the "free" water of the saturated concrete could amount to 5 percent or more and could be much higher in partially saturated concrete.

It is to be noted that the salt content varies greatly from point to point and thus can give rise to powerful corrosion cells of the differential concentration type.

Early in the investigation a number of laboratory experiments using fragments of

TABLE 6
CHLORIDES IN SECTIONS SAWED FROM 2-INCH CORES, SPAN 411
 (Chloride Ion in Percentage by Weight of Air-Dry Mortar)

Member	Position ^a					
	1	2	3	4	5	6
Beams	0.11	0.06	0.05	0.02	0.01	0.01
Caps	0.09	0.07	0.08	-	0.06	0.05
Piles	0.29	0.21	0.12	0.15	0.06	-
Piles	0.31	0.23	0.15	0.08	0.04	0.04

^aPosition of 1-in. sections in relation to outer surface of member, 1 designating outer section.

cores from the bridge, demonstrated that the concrete could function as an electrolyte for the steel. A flow of current between two bars embedded several inches apart in a core was measured to be 0.7 microamp when the concrete was in an air-dry condition. When the concrete was dampened the current increased to 10 microamp. Electrical resistivity of a number of specimens in air-dry condition varied from 270,000 to 660,000 ohm-cm. When damp, the resistivity dropped to the range of 21,600 to 44,200 ohm-cm. The latter values are of the order reported for soils at locations where buried steel pipes have corroded.

In summary, with respect to the concrete of the bridge as a factor in promoting corrosion of the steel, it has been shown that its pervious character and the absorbed sea salts are of primary importance. It has been shown that the concrete can function as an electrolyte and that the activity of corrosion cells increases greatly when the concrete is damp. A limited degree of sulfate attack, although constituting a problem, is not believed of itself to have contributed to corrosion of the reinforcement.

POTENTIAL SURVEYS OF BRIDGE UNITS

Preliminary surveys of electrical potentials existing in a few bridge members and in abandoned piles found lying within the tidal range on the shore were made by connecting a voltmeter to the reinforcing steel and thence to a copper sulfate half-cell, the porous bottom of which was placed in contact with the surface of the concrete. The half-cell was moved from point to point and the potential difference to the steel was recorded.

TABLE 7
CHLORIDES IN FRAGMENTS OF CONCRETE ADJACENT TO
REINFORCING STEEL
 (Chloride Ion in Percent by Weight of Air-Dry Mortar)

Member	Condition of Reinforcement	
	Corroded	Not Corroded
Beams, lower face	0.07, 0.07, 0.10, 0.16, 0.23, 0.24, 0.35, 0.60	0.05
Caps, lower face	0.40, 0.86	0.16
Piles	0.50	
Old shotcrete on piles	0.02, 0.04, 0.17, 0.18	

After making measurements over a systematic grid, a series of curves or contours of equal potential was plotted. The equipotential contours, if of positive polarity, were found to close around anodic areas in which it was found by inspection that the embedded steel had corroded. Where the contours of negative polarity closed, a cathodic area was indicated. The distances between anodic and cathodic centers varied from 2 to 10 ft. Many checks have confirmed the presence of corrosion at indicated anodic areas, both in cracked and uncracked concrete.

Later, a systematic survey was made over portions of 13 spans spaced throughout the length of the bridge at locations considered to represent variations in degree of distress and of repair and maintenance steps that had been performed. In some cases

TABLE 8
AVERAGE TEST VALUES IN SPANS, ARRANGED IN ORDER OF DETERIORATION

Span No.	Percentage of Cracking ^a	Comp. Stress, psi	Beam Cores					Pulse Vel., fps	Resist. in Anodic Areas, ohm-cm
			E $\times 10^{-6}$	Weight, pcf	Absorption, %	MgO, % ^b	Cl, % ^c		
129	100	3,720	2.60	151.4	6.2	6.5	0.08	12,800	26,000
202	95	3,330	2.80	148.6	5.7	8.1	0.10	12,100	27,000
275	80	2,980	2.23	149.6	6.6	8.3	0.08	12,200	
335	75	3,950	2.58	152.1	6.3	7.1	0.10	12,600	23,000
Avg	88	3,490	2.55	150.4	6.2	7.5	0.09	12,400	25,000
70	60	3,410	2.86	149.9	6.3	6.9	0.03	12,800	28,000
553	50	3,310	2.86	150.5	6.5	5.8	0.20	12,700	35,000
560	45	3,560	2.67	149.3	7.0	7.3	0.23	12,900	46,000
Avg	52	3,430	2.80	149.9	6.6	6.7	0.15	12,800	36,000
495	20	3,830	2.78	150.5	6.3	7.7	0.08	13,100	45,000
925	18	3,770	2.94	150.9	6.1	8.0	0.10	13,500	46,000
1,045	12	4,000		152.3	6.5	7.8	0.11	13,000	54,000
1,163	3	4,270	2.56	152.2	5.7	6.9	0.08	13,900	61,000
163	0	2,950	2.35	149.8	6.4	8.1	0.06	12,800	67,000
Avg	10	3,760	2.66	151.1	6.2	7.7	0.08	13,300	55,000

^a Relative length of longitudinal cracks along bottom of beams.

^b Percent by weight of cement.

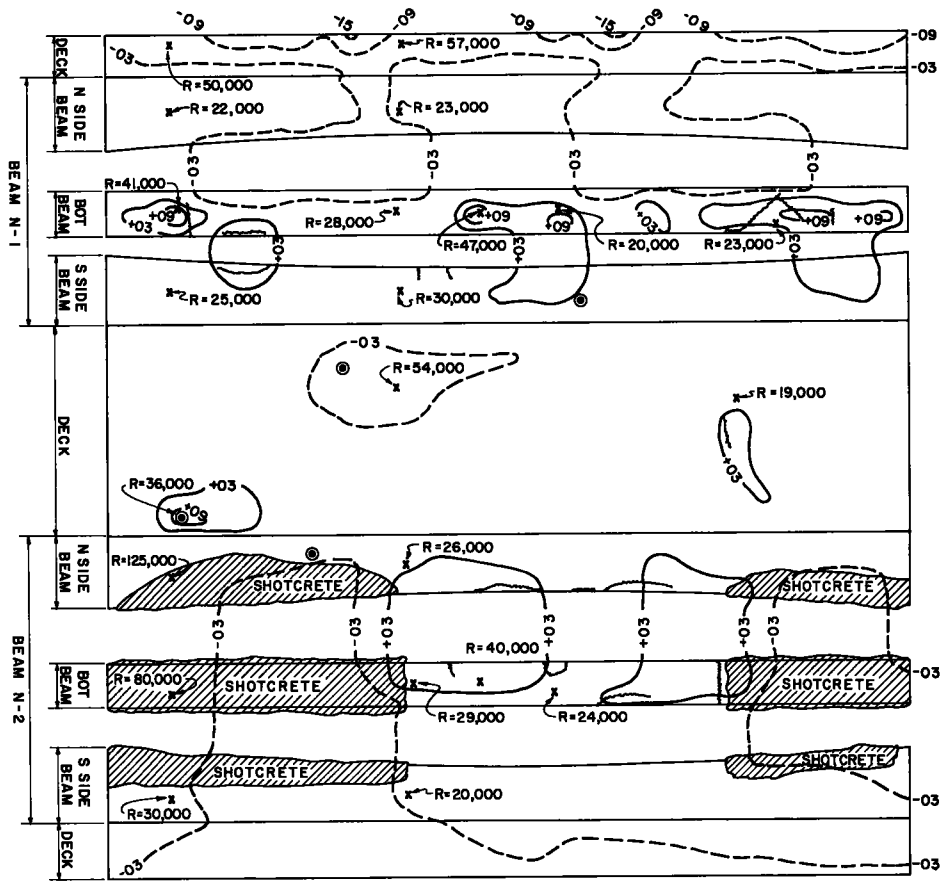
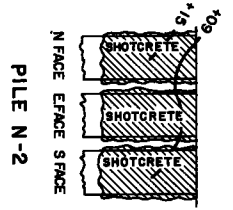
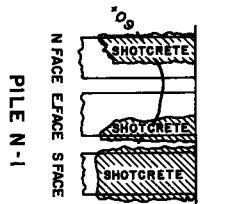
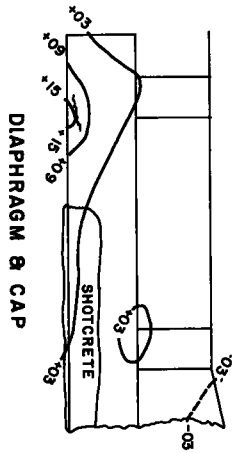
^c Percent by weight of air-dry mortar.

the voltmeter was connected to two copper sulfate half-cells, one of which remained in a fixed position. The results are called surface potentials and can be adjusted to indicate polarity to the steel itself. Data of these measurements are used as the basis of the discussion to follow.

Charts depicting the condition at three spans are shown in Figures 16, 17, and 18 which show the developed surface of the bridge units, visible cracks in the concrete, areas that have been repaired with shotcrete, equipotential contours, and resistivity of the concrete at certain locations.

Figure 16 represents the conditions found in span 70. About 60 percent of the bottom on one beam was repaired with shotcrete in 1951. Cracks have appeared in the remainder of this beam and also in the adjacent beam. These cracks presumably occurred since the beams were repaired. The equipotential contours indicate the measured electrical potentials. The corrosion of the steel occurs in the anodic areas.

It will be noted that the steel in repaired areas is now cathodic to the steel in the



- LEGEND**
- ANODIC VOLTAGE
 - - - CATHODIC VOLTAGE
 - - - CRACKS
 - ⊙ CONCRETE CORES
 - R RESISTIVITY

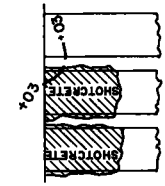
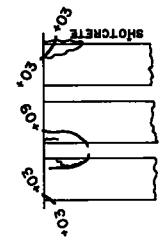


Figure 16. Equipotential contours, Span 70.

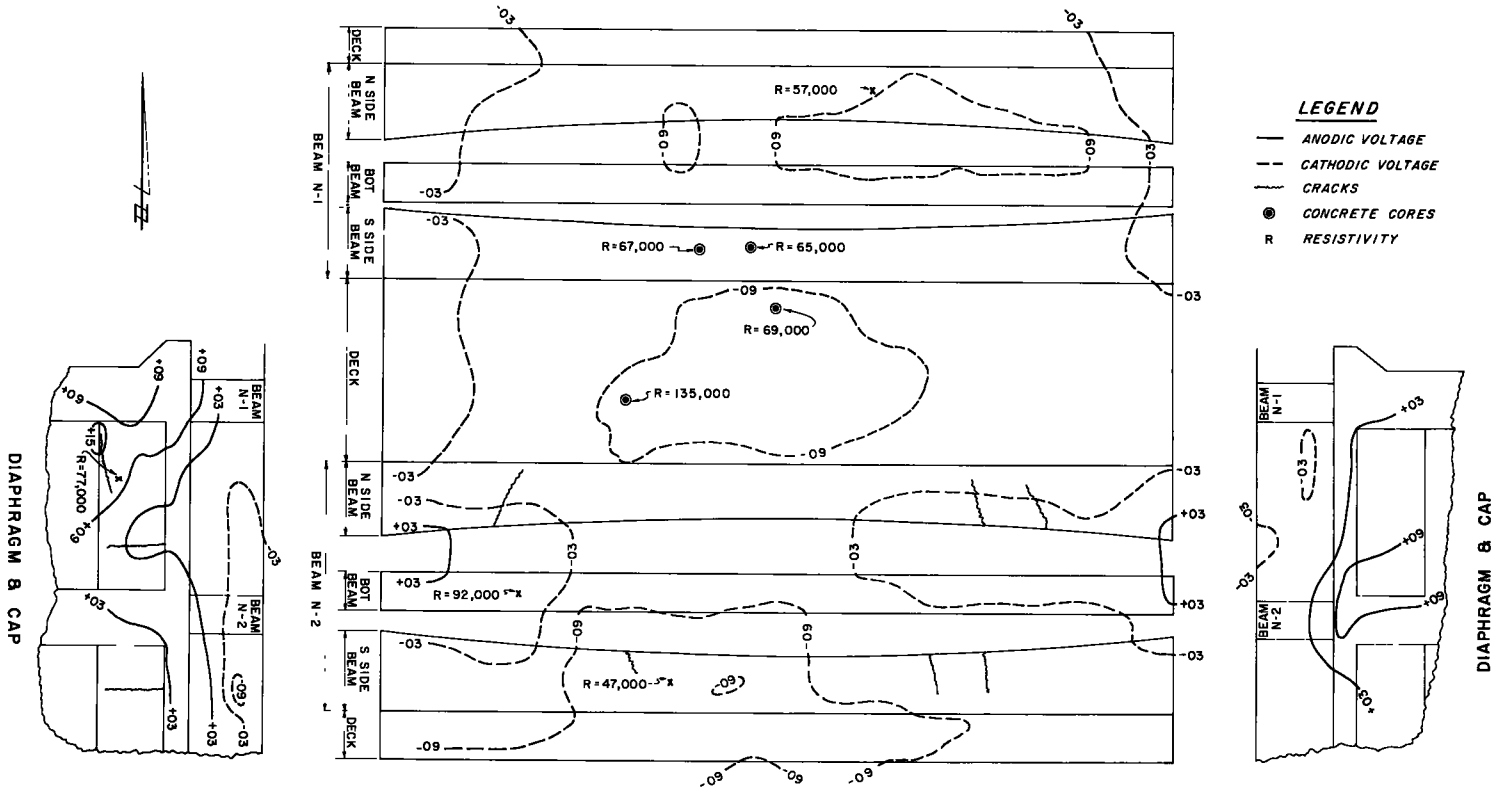


Figure 17. Equipotential contours, Span 163.

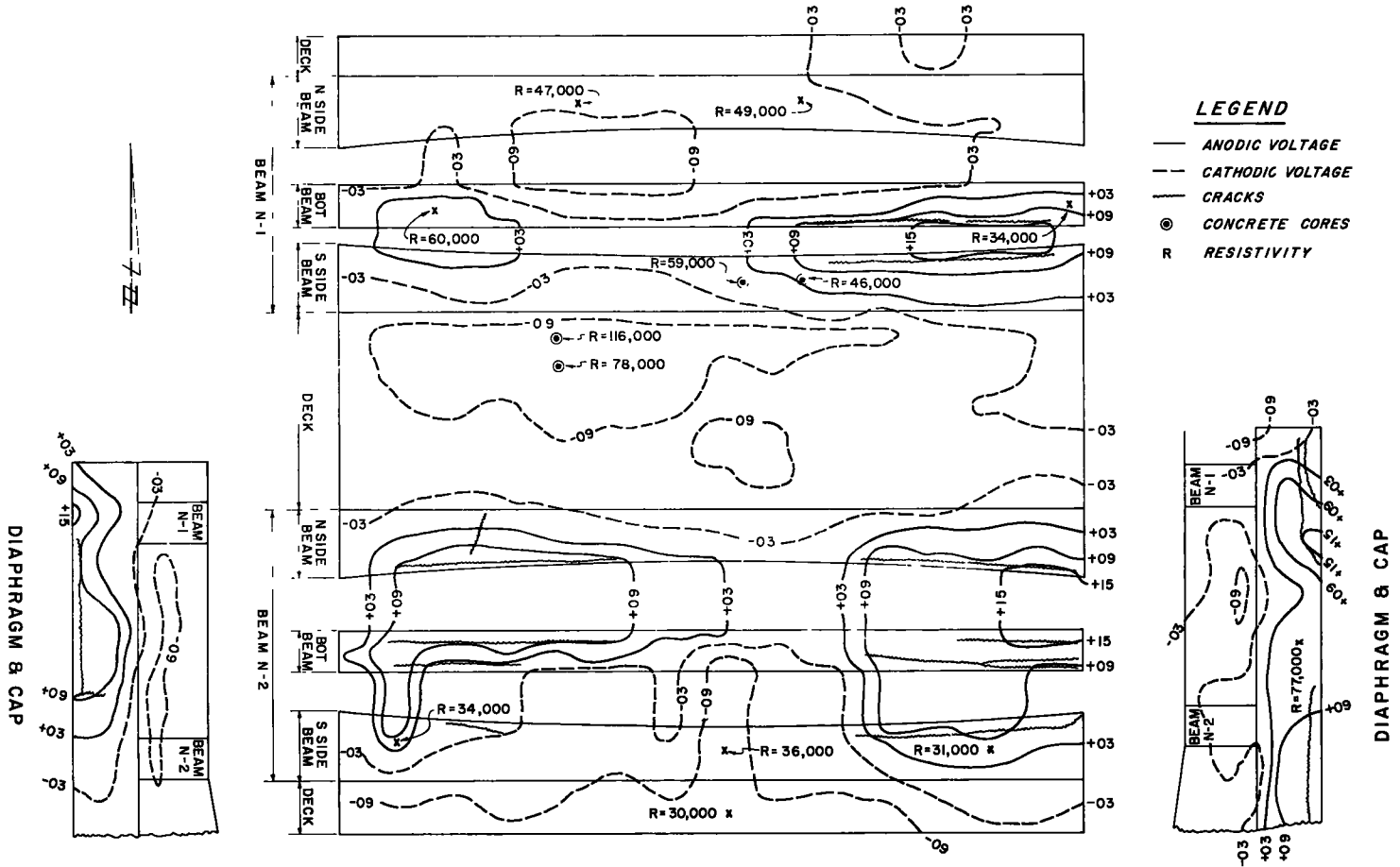


Figure 18. Equipotential contours, Span 553.

original concrete; presumably the steel that is now encased in shotcrete was in an anodic area before the repair. Anodic areas are present at locations adjacent to the shotcrete and elsewhere in the beams where cracking has occurred. Two anodic areas have developed in the deck slab and may be the result of creating cathodic areas within the shotcrete or the loss of the previously anodic location. Anodes are present in the caps and piles, including portions that have been repaired with shotcrete. The average resistivity of the concrete in the anodic areas is 30,000 ohm-cm, and in the cathodic areas 54,000 ohm-cm.

Figure 17 represents span 163, which is adjacent to the steel lift span section and is higher above the bay than other concrete spans. The deck unit is free from longitudinal cracking, although there are faint or embryo anodes starting at the ends of one beam. Otherwise, no strong corrosion cells are indicated by the electrical measurements in this deck unit. There are strong anodes in the diaphragms and caps, and

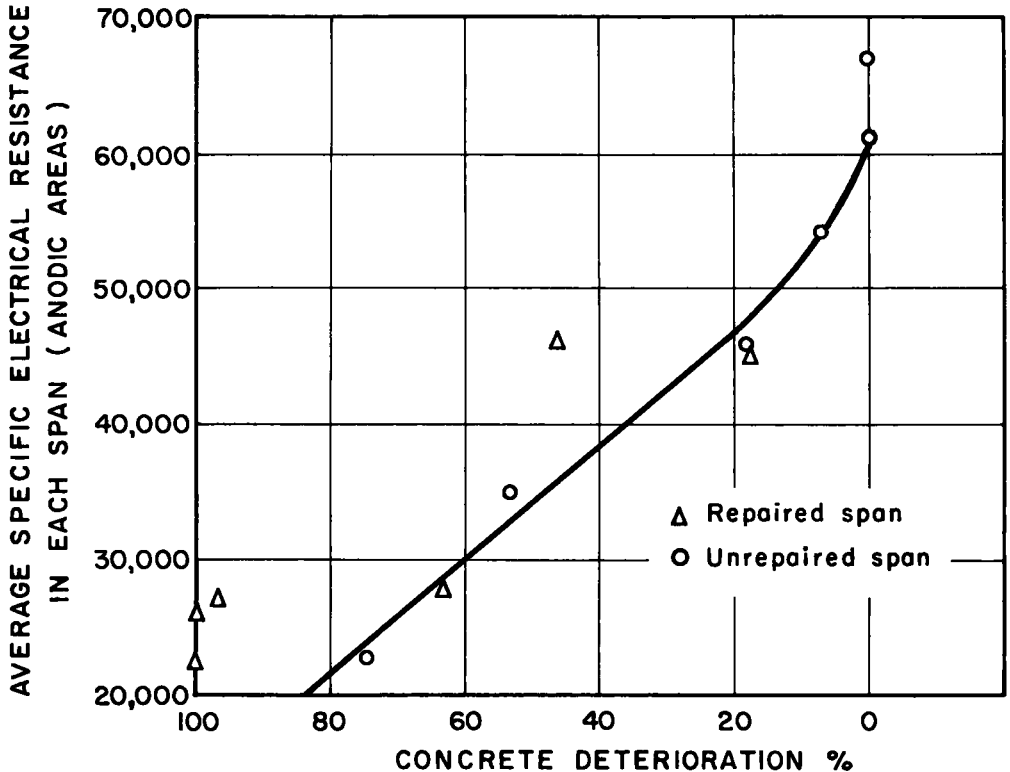


Figure 19. Specific electrical resistance against concrete deterioration.

cracks in the concrete were observed in one diaphragm. The resistivity, measured in cathodic areas, averages 76,000 ohm-cm.

Figure 18 depicts span 553, which is representative of a badly deteriorated span that has not been repaired with shotcrete. The field crews observed longitudinal cracks in the concrete covering approximately 50 percent of the length of the underside of the beams. There are strong anodes where cracking has occurred, but elsewhere the beams are neutral or only weakly anodic. The entire deck slab is cathodic. The anodes in the caps appear to be related to the cathodic areas in the diaphragms. The resistivity in anodic areas averages 37,000 ohm-cm, and in the cathodic areas 50,000 ohm-cm.

The algebraic difference in potential between anode and cathode represents the driving force of the corrosion cell. In the spans illustrated, surface potentials in the deck unit indicate that a maximum surface driving force of 0.4v was responsible for the corrosion of the reinforcing steel.

Woodworth (23) has constructed differential concentration cells consisting of steel plates embedded in blocks of concrete containing sea salt of varying concentration. Corrosion cells were established by electrically connecting two blocks. A potential and flow of current was observed whenever there was a difference in the concentration of salt in the two halves of the cell. Potentials up to 0.35v were measured. These values were thus of the same order of magnitude as those measured on the San Mateo-Hayward Bridge.

The maximum potential that can be generated by such cells probably does not exceed 0.5v. Under relatively constant potentials the resistivity of the concrete should be the determining factor in the activity of the cells.

In Table 8 the spans have been arranged in three groups according to the degree of deterioration. The classification is on the basis of the linear feet of longitudinal cracking along the bottom of the beams, expressed as the percentage of the total length of the

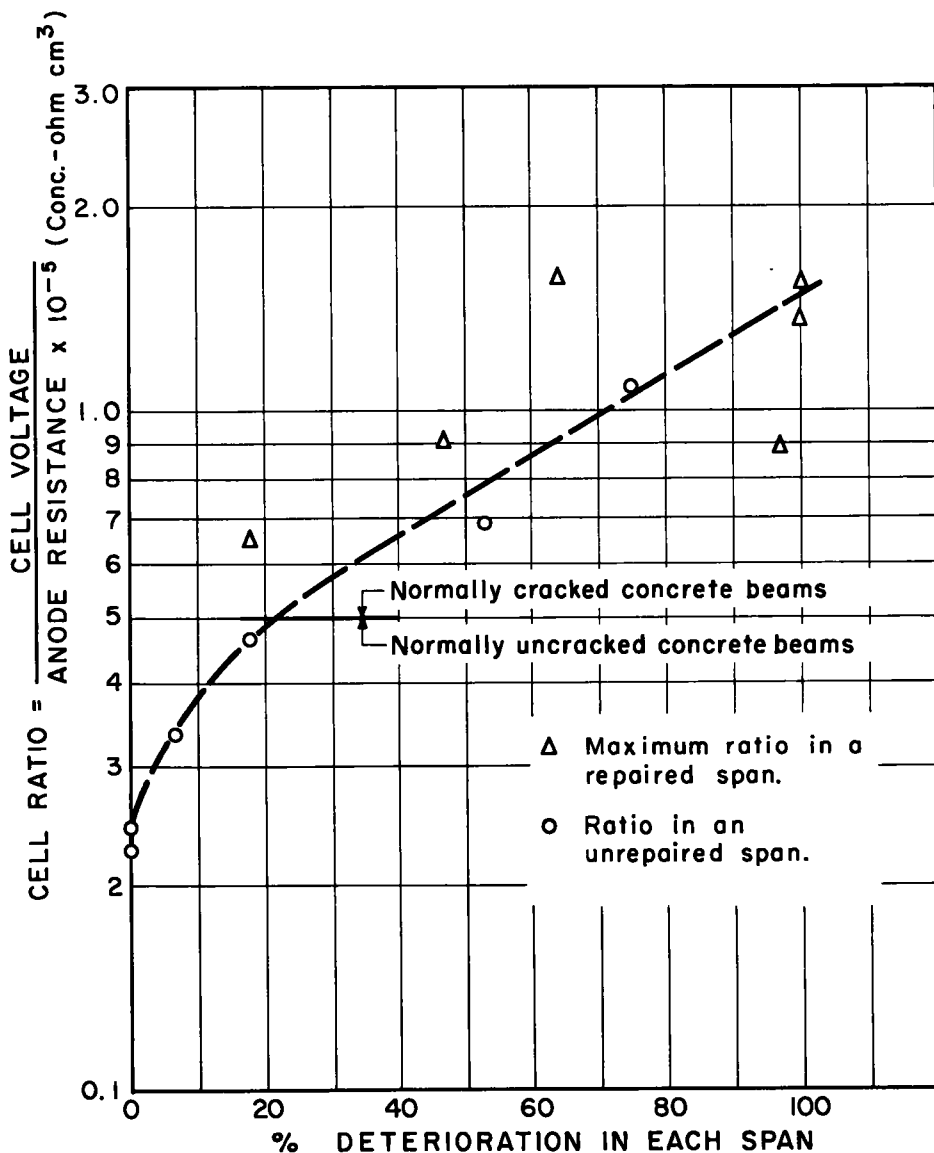


Figure 20. Corrosion cell ratio against concrete deterioration.

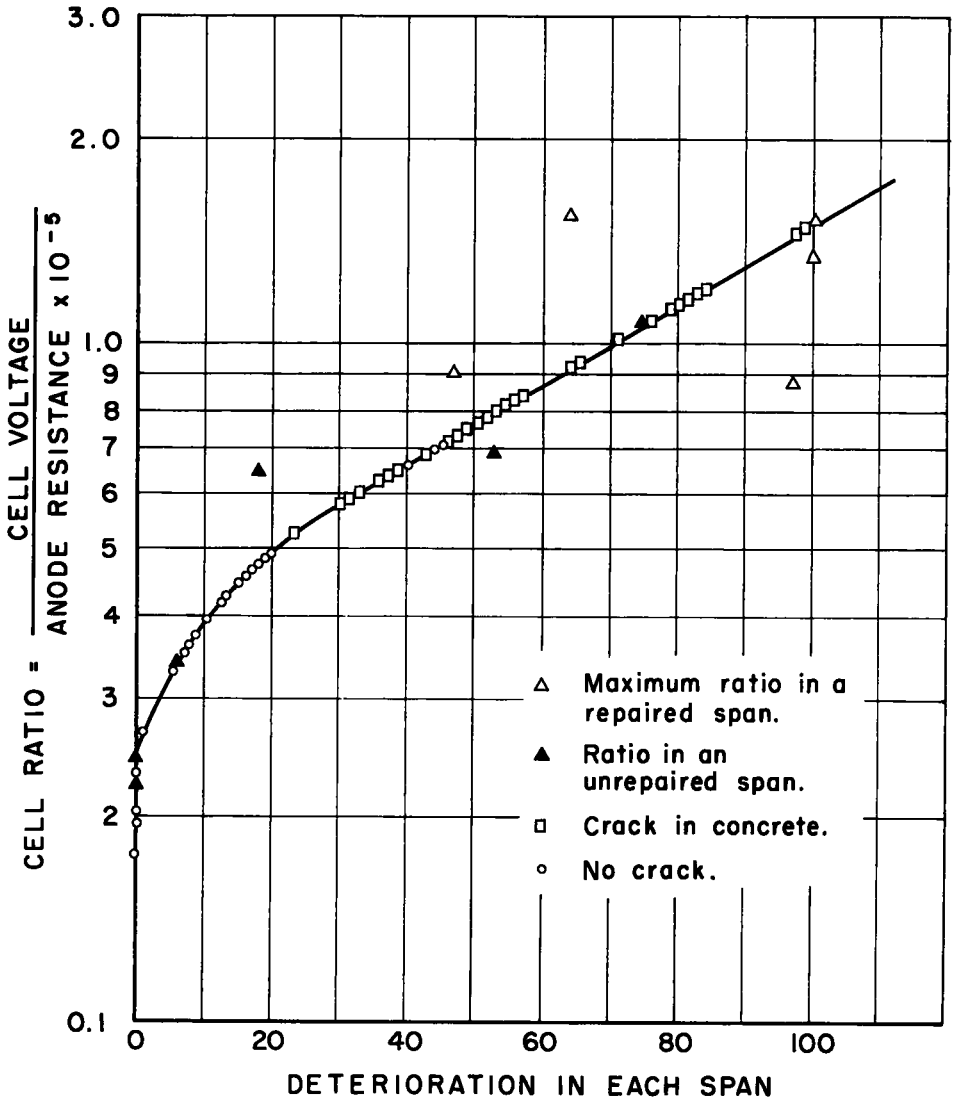


Figure 21. Concrete cracking against corrosion cell ratio.

beams investigated in the span. Span 410 has been omitted because it was repaired shortly before the survey and there is some uncertainty as to the actual amount of cracking that existed prior to repair. The last column of Table 8 shows the resistivity of the concrete in anodic areas. It will be noted that decreasing resistivity is accompanied by increased cracking; also that when the resistivity exceeds about 60,000 ohm-cm there has been little or no cracking.

Figure 19 is a plot of deterioration against resistivity at anodic areas. The general trend is that increasing resistivity is accompanied by decreasing deterioration. Resistivity of 60,000 ohm-cm apparently is required to inhibit or prevent the start of accelerated corrosion in this concrete.

The quantity of reinforcing steel corroded is a function of the amount of current (ampere-hours for example) flowing through the corrosion cell. The rate of current flow cannot be accurately measured in this structure, but can be represented approximately by the potential difference between anodes and cathodes divided by the resistiv-

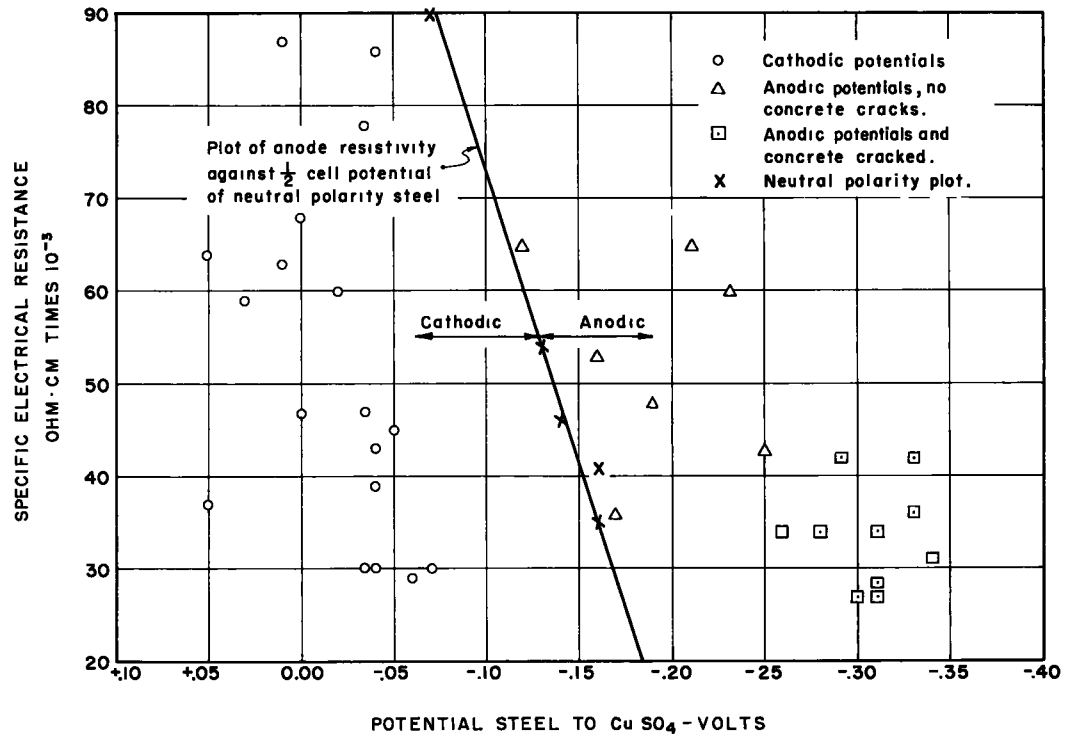


Figure 22. Half-cell potential of steel against resistivity of concrete.

ity. Figure 20 is a plot of deterioration in the several spans against the ratio $\frac{\text{potential}}{\text{resistivity}}$, which for convenience has been designated as the "cell ratio." The value of potential used in the cell ratio is the maximum voltage differential between the cathode (which is generally in the deck) and the anode (which is generally in a beam). The resistivity value of the concrete used in this ratio was measured at anodic areas. The curve indicates that there is a definite relationship between the cell ratio and de-

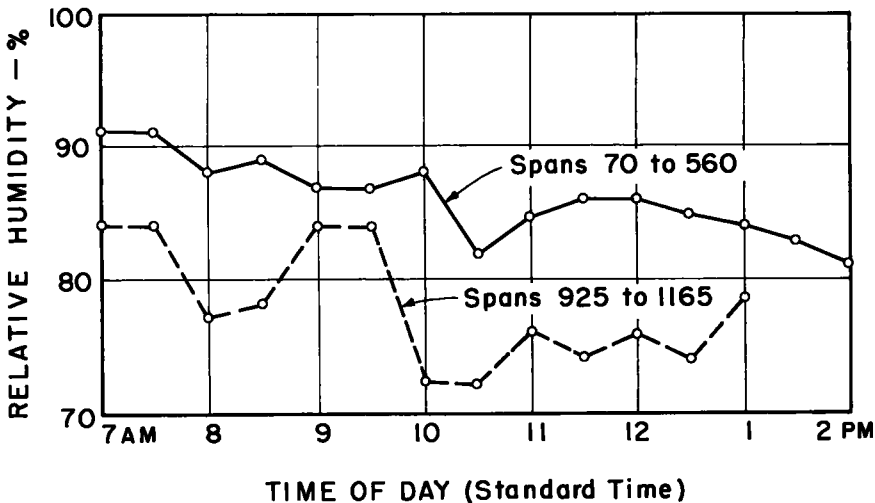


Figure 23. Relative humidity under bridge and shaded from sunlight, July-Oct. 1955.

terioration of the concrete. The curve was determined by the corrosion cells in the deck and beams in the spans under consideration and the shape of the curve may be representative of the deterioration of an average corrosion cell in the bridge. As the curve was drawn from average values, it does not necessarily represent any single cell, but may represent the combined effect of all the cells in a particular span.

The curve of Figure 20 has been reproduced in Figure 21. Each plotted point on the curve represents the cell ratio of an individual corrosion cell. Circles indicate that the concrete has not cracked and squares that it has cracked. No further distinction has been made as to degree of deterioration. It will be noted that no cracking has occurred when the cell ratio is less than 0.5. With two exceptions, the concrete has cracked when the cell ratio exceeds 0.5. This value appears to be of critical significance for this structure at the present time.

In some of the spans potentials were measured by direct connection to the reinforcing steel; in others, the measurements were related to a fixed potential point on the surface of the concrete. In the later case, differences in potential were measured and the contours were plotted. The observed potential relative to steel was not known when this method was used. Five of the spans were in the first category and Figure 22 is presented to illustrate the relationships obtained in these spans. The crosses in this figure represent the apparent neutral half-cell potential of the steel to a copper sulfate reference cell. The sloping line represents the neutral potential of the steel for varying amounts of resistivity (specific electrical resistance of the concrete) as determined by the position of the crosses. Cathodic potentials fall to the left of the curve and anodic potentials to the right.

The chart shows that cracking in these spans occurred only when the anode resistivity was below 45,000 ohm-cm, but that there are two points where the resistivity was this low but in uncracked concrete. The difference in behavior seems to be related to the excess in the negative potential from the neutral point of the steel.

MOISTURE AS A FACTOR IN CORROSION

Variations in the amount of absorbed water in the concrete cause variations in the concentration of dissolved salts and thus can establish differential concentration cells, even though the percentage of salt by weight of the concrete does not vary.

Test results have been cited that show the resistivity of air-dry concrete to be 10 or more times that of the same concrete when moist or wet.

The prevailing level of moisture in the concrete, then, is an important factor that influences the activity of the cells and the consequent rate of corrosion of the steel. It has been observed that the deterioration rate of the structure is less at the east end. In the spans surveyed, the concrete at the east end had higher resistivity. It has been reported that fog is more prevalent toward the west shore of the bay. Weather bureau records indicate a higher level of relative humidity on the west shore.

The relative humidity beneath a particular span was recorded during the days that potential surveys were under way. Average results are shown in Figure 23 for measurements during July, August and September at spans 70 to 560 at the west end, and during October at spans 925 to 1165 at the east end. The average relative humidity at the east end is 9 percentage points lower than that at the west end. Although the determinations were not made concurrently, Weather Bureau records indicate that a differential of this order exists.

The level of absorbed moisture within the concrete must be in equilibrium with the prevailing or average relative humidity of the surrounding atmosphere. Steps are under way to install humidity sensing elements in drilled holes at various locations in the bridge.

An exception to the general rule that deterioration is more severe toward the west end of the structure is in the spans forming the approaches to the steel lift span near the west end. The approach spans are inclined and reach an elevation 7 ft higher than the general level of the bridge. Figure 17 shows the condition of span 163, which is adjacent to the steel truss. There are no cracks in the deck unit of this span, resistivity is high, and the deck units have only traces of anodic conditions. It is possible that

the concrete is drier because of greater elevation above the water. Humidity sensing elements are to be installed at this location.

SHOTCRETE REPAIRS

Part I discussed in considerable detail the precautions necessary to secure a shotcrete repair of permanent character. Many instances of ruptured shotcrete more than 5 years old verify the need of careful planning and execution of the work. Removal of at least 2 in. of concrete behind the bars not only makes thorough cleaning more certain, but also provides additional protection against the salts present in the old concrete. However, to characterize even the most carefully executed shotcrete as being practically "permanent" requires more optimism than past experience and recent test results appear to warrant.

Table 7 shows that shotcrete has, in time, absorbed substantial amounts of salt. Equipotential surveys have shown that in some instances anodic areas have developed within shotcrete repairs. The piles illustrated in Figure 16 are examples of this. The resistivity of newly placed shotcrete has been measured to be 150,000 ohm-cm. In older shotcrete the resistivity has dropped seriously. Of the spans investigated, eight contained shotcrete of various ages. In four of these the average measured resistivity of the shotcrete was from 15,000 to 29,000 ohm-cm. These results point to the eventual corrosion of shotcrete-encased reinforcement. The length of time that will be required for the indicated corrosion cell action to reach destructive proportions cannot be estimated from present experience.

Attention is called to the statements of Part I with respect to the frequent development of destructive corrosion in areas of unrepaired concrete adjacent to, or in the vicinity of, shotcrete repairs and the indicated possibility that virtually the entire mass of concrete above the low water line may in time require repair. Part I also called at-

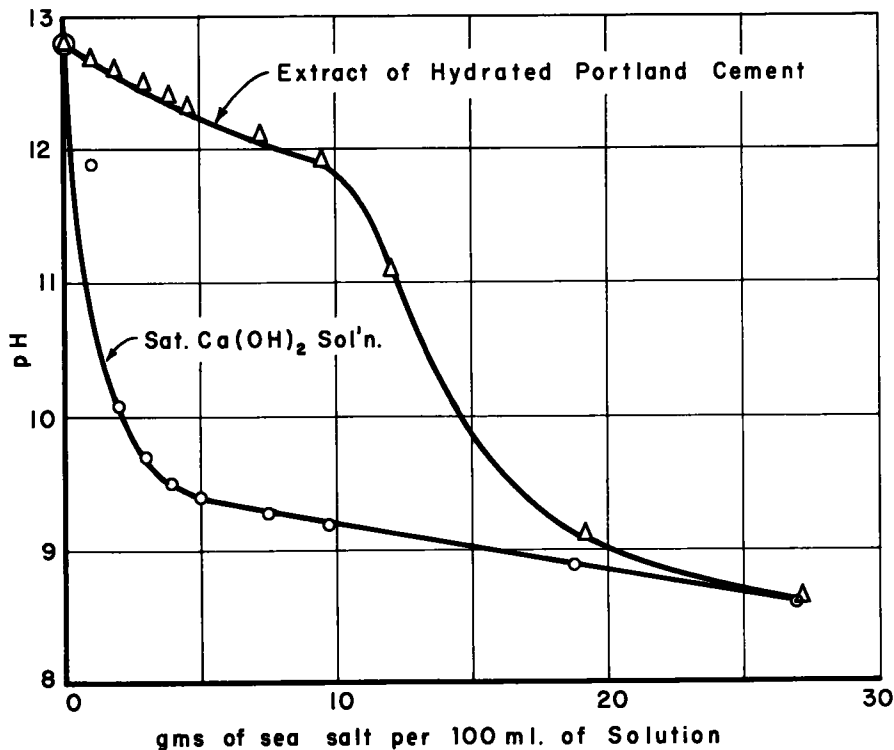


Figure 24. Effect of sea salt on pH of calcium hydroxide solution and hydrated Portland cement extract.

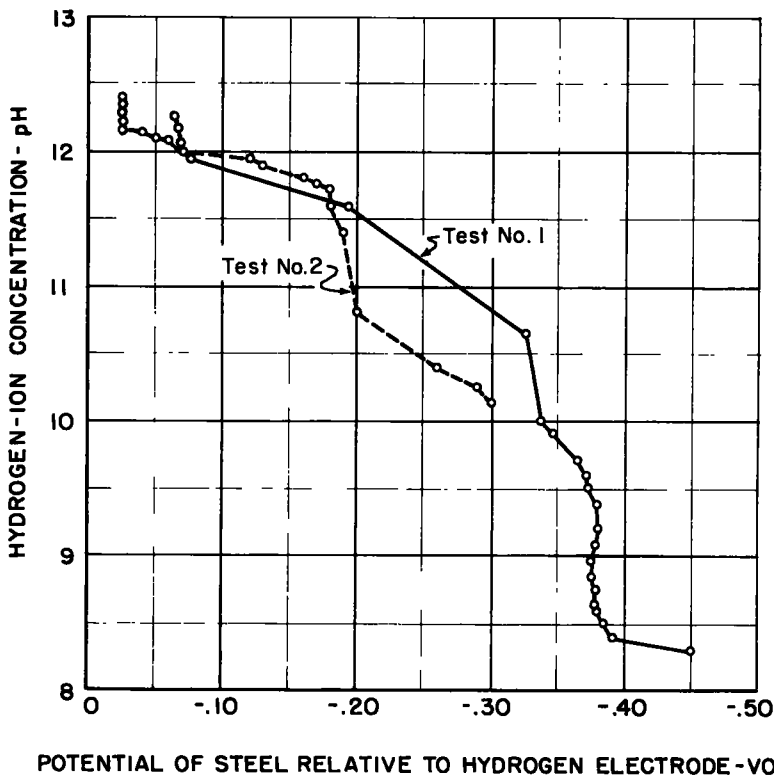


Figure 25. Potential of steel in cement extract solution of variable pH by addition of sea salts.

tention to the possible development of corrosion cells as a result of interaction between shotcrete repairs that have been placed at different times.

The findings and discussion previously presented point to the possibility that methods of treatment other than by shotcrete repair may prove to be more economical on a long-term basis.

LABORATORY EXPERIMENTS

During the course of the field investigation, a number of experiments have been initiated in the laboratory using specimens in which predetermined amounts of sea salt were added at the time of mixing. These tests make it possible to explore the effects of moisture and salt content on corrosion of steel over a greater range and under better control than can be obtained from measurements on the bridge.

Water contained in concrete is considered to be saturated with respect to calcium hydroxide and to contain a relatively constant weight of this compound at all times. The concentration of sea salts in the water depends on the amount contained in the concrete and the amount of water available as a solvent. In concrete such as that of the San Mateo Hayward Bridge, the ratio of sea salt to calcium hydroxide in solution may vary between wide limits. For this reason the pH of the water may vary.

Examples of the possible range in pH are shown by Figure 24. Evaporated sea salt was added to the solutions indicated. The extract of hydrated portland cement was prepared from a hardened neat cement paste of Type I cement, containing 1.10 percent alkalis, similar to that used in the bridge. The hardened paste was oven dried and granulated. It was continuously agitated for three days in sufficient water to bring the total amount to $7\frac{1}{2}$ gal. per sack of cement. After filtering, varying amounts of sea salt and a small amount of hydrated cement were added to portions of the extract.

pH values were determined at intervals up to 7 days, at which time they were constant.

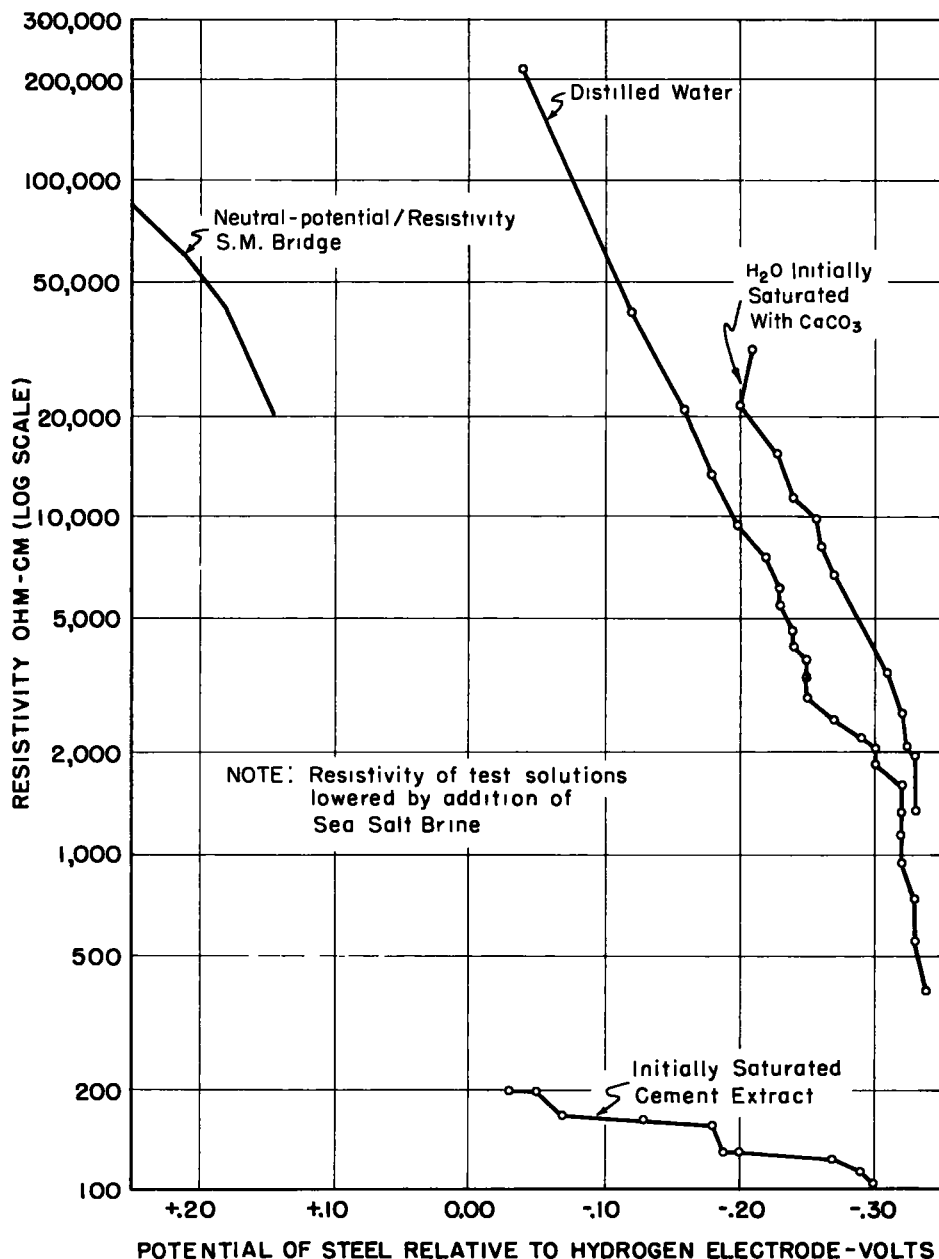


Figure 26. Potential of steel in solutions of variable resistivity by addition of sea salts.

Each sample was then analyzed for chlorides and calcium hydroxide in solution. The data (Fig. 24) represent only one series of tests and the shape of the curve has not been verified.

Because of loss of alkalis through leaching, it is probable that a curve representing the concrete of the San Mateo-Hayward Bridge would be between the two experimental curves. It is probable that the contained water could be reduced to pH 10 if the concentration of sea salt were of the order of 5 to 10 percent. Such a concentration is possible in partially dried concrete containing sea salt in the quantities found in the bridge.

The effect of sea salt-cement extract solutions of varying pH on the half-cell poten-

tial of a steel electrode relative to the calomel-cell has been determined. Adjusted results relative to the hydrogen cell are shown in Figure 25. In an extract containing no sea salt and having a pH of 12.5 the half-cell potential of the steel was approximately that of the hydrogen electrode. At pH 10, it was about -0.43v relative to the hydrogen cell. This value is close to that of iron going into solution in the ferrous state. The results indicate that salt can destroy the protective effect of normal concrete against the corrosion of steel.

Figure 26 is presented as experimental evidence that there is an interrelationship

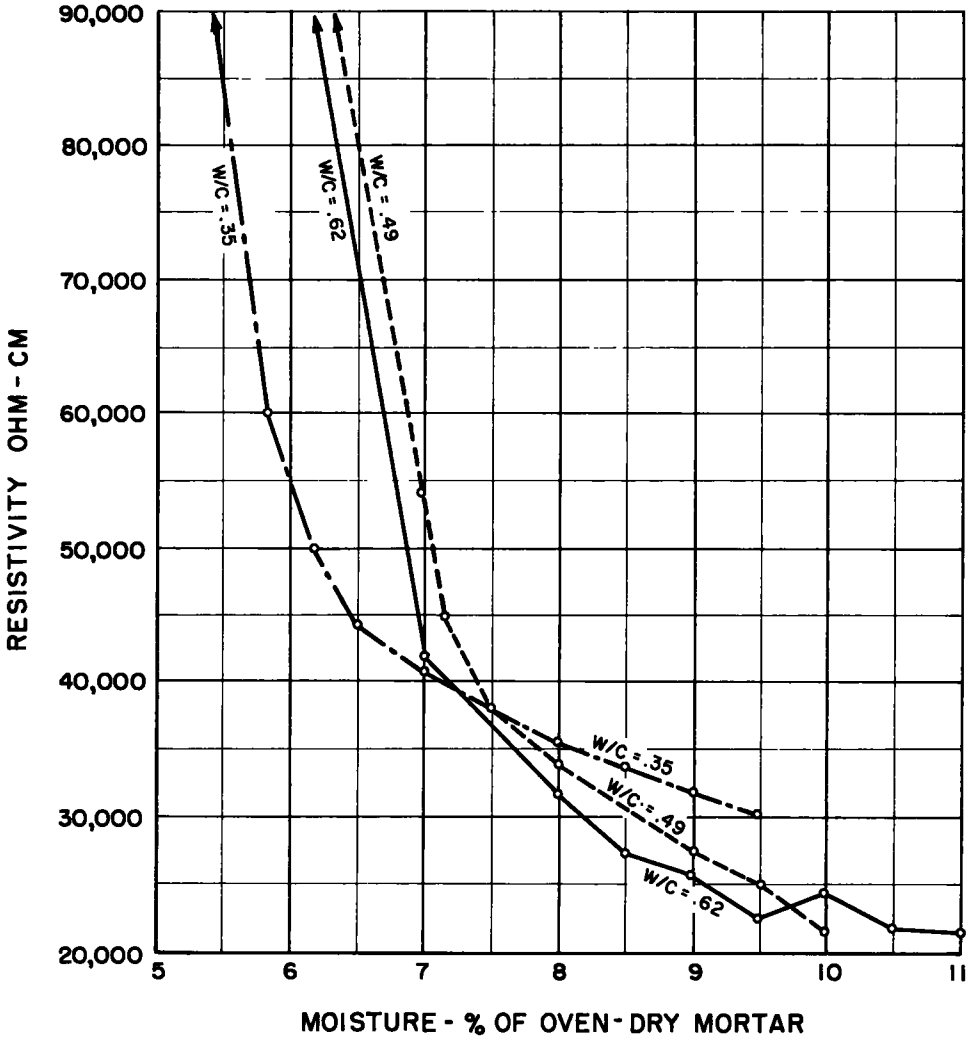


Figure 27. The effect of water cement ratio on the resistivity of mortar at various moisture contents (all salt contents averaged).

between salt content, electrode potential, and resistivity, and thus suggests the possibility of using measurements of two of these factors to estimate the third. It is evident however, that calibration curves applying to the particular materials involved must be established before the unknown value can be estimated with accuracy.

The discussion so far has dealt with water extracts of cement. Further experiments have been made with specimens consisting of steel electrodes in mortar of three water cement ratios, containing sea salt in amounts of 0.0, 0.10, 0.25 and 0.60 percent ex-

pressed as chloride ion in percentage by weight of air-dry mortar. The specimens were allowed to dry slowly in air and at intervals the loss in weight and resistivity was determined. Finally, the specimens were oven dried. Figure 27 shows that water-cement ratio, of itself, has less effect on resistivity than does change in moisture content due to drying. Likewise, Figure 28 shows that salt content is of minor effect on resistivity unless it exceeds 0.25 percent. The relative degree of saturation of the concrete is the major factor that determines its resistivity.

These experiments make it possible to estimate roughly the degree of dryness at

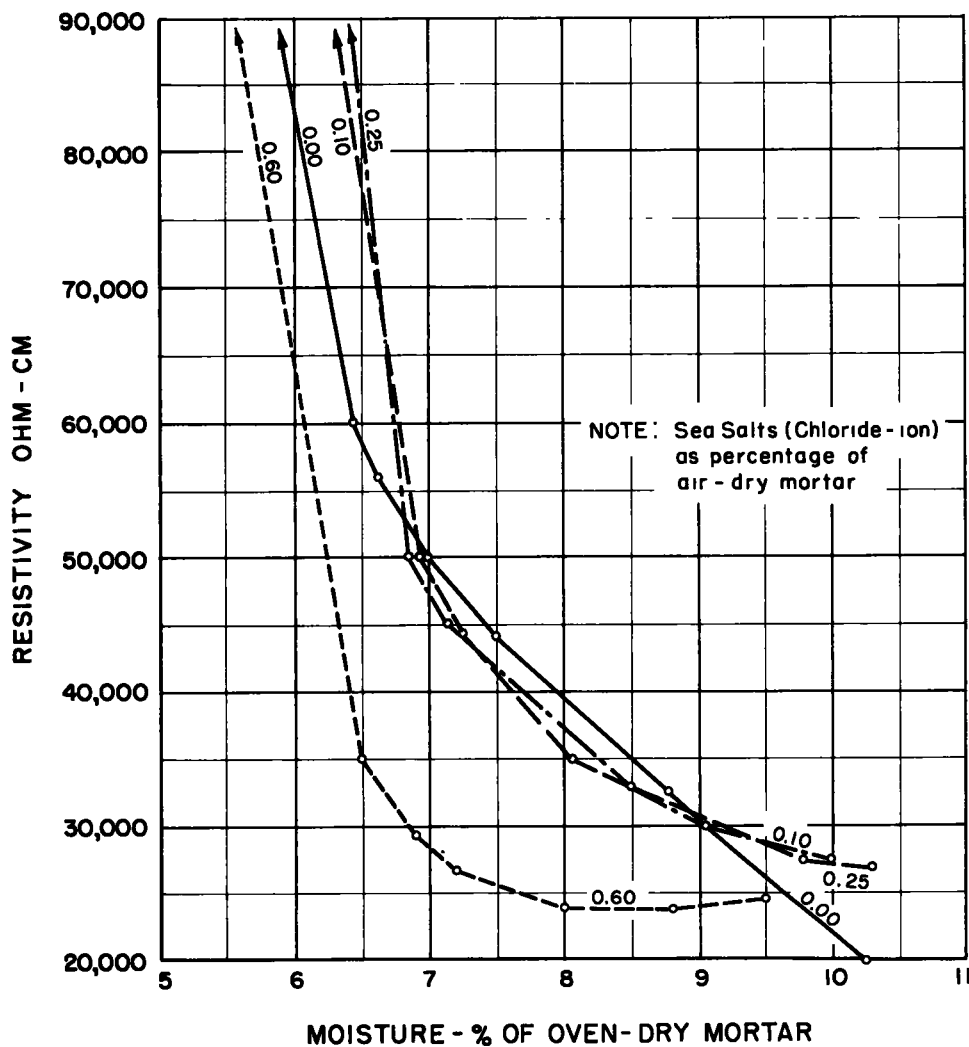


Figure 28. The effect of sea salts on the resistivity of mortar at various moisture contents (all water-cement ratios averaged).

which concrete containing sea salt will inhibit corrosion of the reinforcing steel. It will be recalled that in the discussion of resistivity and deterioration (Fig. 19) it was stated that cracks had not appeared in concrete of the bridge when its resistivity exceeded about 60,000 ohm-cm. Figures 27 and 28 indicate that in the cement mortar test specimens, the resistivity exceeded 60,000 ohm-cm when the moisture content was less than about 6 or 7 percent above its oven dry content. Since mortar ordinarily comprises about one-half the weight of concrete, it is indicated that if concrete containing salt does not contain more than 3 to $3\frac{1}{2}$ percent of moisture above its oven dry

TABLE 9
INITIAL ELECTRICAL RESISTANCE (IN OHMS)
OF IMPRESSED CURRENT CELLS

Chloride Ion, Percent	Impressed Voltage		
	10	2	0.5
0.60	300	550	4,500
0.25	410	1,700	12,000
0.10	340	1,800	17,000
0.00	280	2,000	68,000

content, corrosion will be greatly retarded or possibly prevented.

A series of 24 4- by 4- by 4 $\frac{1}{2}$ -in. blocks of mortar containing two $\frac{1}{4}$ - by 2-in. steel electrodes separated 2 in. was made up with 0.0, 0.10, 0.25 and 0.60 percent of sea salt (expressed as chloride ion) added at the time of mixing. After moist curing for 6 months, the specimens have been subjected to impressed direct current of 10, 2, and 0.5v while being maintained in a substantially saturated condition. Table 9 shows the initial internal electrical resistance of the cells under impressed current. It will be noted that under 10v of impressed current the internal resistance was nearly constant for all salt contents. However, the resistance increased greatly as the impressed voltage was decreased, and for these lower potentials was greatly diminished by increasing salt content. Under 0.5v, which approximates the maximum due to galvanic action

TABLE 10
IMPRESSED CURRENT TEST CELLS^a

Chloride-Ion, % ^b	10-Volt Series		2-Volt Series		0.5-Volt Series	
	Current Drawn, amp-days	Cell Reaction	Current Drawn, amp-days	Cell Reaction	Current Drawn, amp-days	Cell Reaction
0.60	0.129	Broke in 4 days	0.049	Broke at 11 days	0.016	Broke at 67 days
0.60	0.121	Broke in 4 days	0.052	Broke at 11 days	0.015	Broke at 41 days
0.25	0.072	Broke in 4 days	0.033	Broke at 18 days	0.038	No break 188 days
0.25	0.085	Broke in 4 days	0.052	Broke at 18 days	0.027	No break 188 days
0.10	0.059	Broke in 4 days	0.165	No break 188 days	0.00005	No break 188 days
0.10	0.064	Broke in 4 days	0.213	Broke at 178 days	0.00008	No break 188 days
0.00	4.31	No break 188 days	0.154	No break 188 days	0.00002	No break 188 days
0.00	4.44	No break 188 days	0.158	No break 188 days	0.00010	No break 188 days

^aElectrodes $\frac{1}{4}$ -in. steel, 2 in. wide and imbedded 3 in. in concrete.

^bPercent by weight of air-dry mortar.

in the San Mateo-Hayward Bridge, the amount of current flowing through the cell was increased greatly by the sea salt.

It was observed that free water containing iron compounds appeared on the surface of the concrete at the anodic electrode of the cells containing salt and formed in sufficient quantity to run over and stain the vertical faces of the cell. More liquid accumulated under the higher voltages. These observations point to the conclusion that iron stains originating at the reinforcement are evidence of the presence of salt in the concrete.

Table 10 shows the number of days of impressed current at which the cells ruptured and the accumulated ampere-days of current to the time of rupture, or up to 180 days in intact blocks. It is to be noted that none of the salt-free cells ruptured in this period. Under 0.5v only the cells containing 0.60 percent salt ruptured.

A cell with two electrodes was constructed in which one-half contained 0.10 percent and the other half, 0.60 percent sea salt (as chloride ion). When the electrodes were connected there was a flow of current. The electrode in the area of greater salt content was the corroding or anodic electrode. When the cathodic side (lower salt) was placed in water and the anodic side was allowed to dry, less current flowed. When the anodic side was placed in water and the cathodic side was allowed to dry, the electrical current increased. Woodworth (23) performed many experiments with cells of differential salt content with results similar to that previously described. When access of oxygen to the cathode was restricted, the cell became dead or its action was greatly impeded.

The laboratory experiments taken as a whole, and considered in the light of field observations and tests, afford conclusive evidence that corrosion of reinforcing steel in the San Mateo-Hayward Bridge has resulted from unequal concentrations of sea salts, which have produced corrosion cells of the differential concentration type.

CORROSION IN OTHER CONCRETE STRUCTURES

California has many reinforced concrete structures located close to the ocean or its inlets. A substantial number have developed some degree of corrosion in the reinforcement, accompanied by rupture of the concrete in beams or girders, columns, and piling. Fragments chipped from affected areas of some of these structures have been found to contain salt in amounts of 0.02 to 0.50 percent (expressed as chloride ion by weight of air-dry mortar). In one instance, a reinforced concrete structure 342 ft in length, constructed in 1916, located on the shore of the ocean near the California-Oregon border, developed so much corrosion in the reinforcement that it was considered necessary to replace it with a new structure after 29 years of service.

Repairs have been made with shotcrete in some instances, and cracking has progressed beyond the limits of the repair or has appeared in the shotcrete itself. Corrosion has occurred in concrete of excellent quality and with a 3-in. cover over the reinforcement. With the exception noted, the total distress in these structures is minor compared to the over-all magnitude of deterioration found in the San Mateo-Hayward Bridge, but it offers reason for doubt on the adequacy of present design and construction practices. Criteria for judging the corrosive effect of the environment at a proposed bridge site are needed so that adverse conditions can be taken into account in design.

Deterioration similar to that found in the San Mateo-Hayward Bridge has been reported in structures located in Florida (24), Texas, Malaya and South Africa (23). Halstead and Woodworth (23), after detailed study, attributed the corrosion of reinforcing steel in a group of bridges on the coast of Natal, South Africa, to unequal brine concentration in the body of the concrete which produced destructive potentials; that is, to the same type of action found in the San Mateo-Hayward Bridge. They also commented on the fact that corrosion of this type seems to have been confined to warmer climates and not to have occurred in cooler climates, such as in Germany or Holland.

On the other hand, there is at least one example of corrosion of reinforcement in a cold climate. As part of the Long-Time Study of Cement Performance in Concrete, carefully made reinforced concrete piles were driven in coastal sea waters off Massa-

chusetts, Florida, and California (25). Companion piles were driven in fresh water in New York. The planned cover over the main bars was $1\frac{3}{4}$ to 2 in. In all salt water installations severe cracking has developed in the upper portions of the piles due to rusting of the reinforcing steel. The progress of cracking at the Massachusetts installation has been difficult to follow because of the concurrent spalling due to frost action. Nevertheless, the evidence is clear that cracking in the plane of the reinforcing bars started at an early date. It is to be noted that similar distress has not occurred in fresh water in an equally severe climate in New York.

Although it is difficult to provide perfect protection to reinforcing steel in concrete exposed to sea salt, there are many examples of excellent durability. For example, 1,200 reinforced concrete pile jackets in Pier 17 of the San Francisco harbor installations have given 36 years or more of satisfactory service (27).

CORRECTIVE MEASURES

The investigations described suggest two possible methods of alleviating the corrosion of reinforcing steel in the San Mateo-Hayward Bridge. These are:

1. Apply cathodic protection to nullify the potential differences in the steel.
2. Provide means to exclude oxygen from the cathodic areas of the corrosion cells.

A suggested method of accomplishing cathodic starvation is the application of an oxygen impervious coating to cathodic areas, with and without provision for access of water to maintain the concrete in these regions in a high degree of saturation. Accompanying the treatment of cathodic areas, existing bituminous coatings in the surface of anodic areas are to be removed to encourage access of air to the steel in these locations.

Arrangements are under way to provide both of these treatments experimentally to deck units of selected spans in the bridge. The results will be checked by periodic electrical potential and resistivity surveys.

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