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Bulletin 182

Corrosion of Reinforcing Steel And Repair of Concrete In a Marine Environment

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Causes and Repair of Deterioration to a California Bridge Due to Corrosion of Reinforcing Steel in a Marine Environment

I. Method of Repair

M. W. GEWERTZ, Senior Bridge Engineer
California Division of Highways

The California Division of Highways maintains numerous reinforced concrete bridge structures, on some of which the reinforced concrete has deteriorated from varying causes and in varying degrees. This paper discusses a particular phase of this type of deterioration; namely, deterioration of structural parts as a result of buildup of internal corrosion on steel bar reinforcement in a marine atmosphere and the consequent rupture of the surrounding concrete. The paper describes conditions encountered at the San Mateo-Hayward Bridge, a major transbay structure consisting of a 6.76-mile length of reinforced concrete trestle construction, plus a 1,503-ft length of steel truss construction including a vertical lift drawspan.

This structure, acquired by the state in 1951, some 22 years after its construction, now presents the Bridge Division with its most extensive problem of concrete deterioration from the cause previously mentioned. The extent of the deterioration was considered of sufficient magnitude to justify initiation of a research project to determine the cause and possible cure of the condition.

Part I of this paper discusses the history, character and extent of the deterioration, inspection and estimating procedures prior to repair, repair procedures, basis of contract payment, and costs of repair.

● THE California Division of Highways maintains numerous reinforced concrete bridge structures, some of which in the course of time have developed deterioration of the reinforced concrete from varying causes and in varying degrees. The causes of such deterioration are legion, and may range from defective ingredients or improper preparation of the concrete to adverse climatic exposures, or to various combinations of chemical and physical factors which affect adversely the normally sound condition of the constituent steel and concrete materials.

The Division has in the past been diligent in research into the origin of, and remedy for, defects which have developed in its concrete structures of many varied types. However, the type of deterioration encountered in the San Mateo-Hayward Bridge has not heretofore presented more than a minor maintenance problem in the many structures under the Division's jurisdiction.

This situation was suddenly altered when on September 12, 1951, the State of California purchased from its private owner, The San Francisco Bay Toll Bridge Company, by sale of toll bridge revenue bonds, the existing San Mateo-Hayward Bridge, which had been operated as a toll bridge under private ownership and continues as such under state ownership. At the time of purchase the operation and maintenance of this structure, through established procedure, became the responsibility of the Division. Investigations by the Division prior to purchase disclosed the wide-spread deterioration of the concrete in the structure, and estimated costs of repairing this condition were taken into consideration in arriving at the purchase price to be paid. The agreed purchase

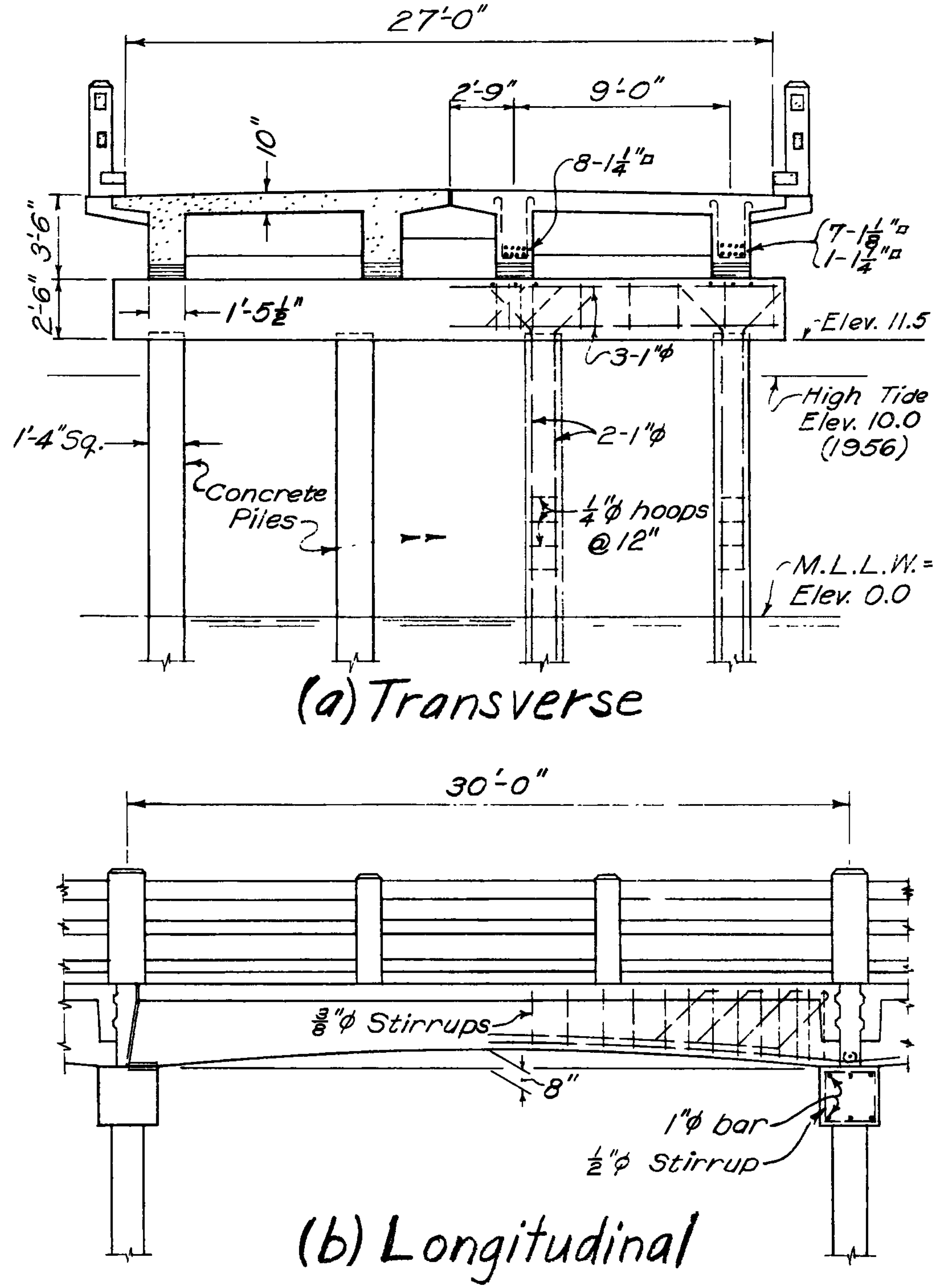

Figure 1. Typical sections of 30-ft span.

price for this structure, including approaches, was $6,000,000.

The purchase of this structure from private ownership explains why the Division finds itself heir to a well-advanced condition of deterioration in a major reinforced concrete structure under its jurisdiction, and has but lately engaged in a research program designed to determine the basic cause of the condition and to develop methods of arresting its progress.

The San Mateo-Hayward Bridge was constructed in 1928-29 by The San Francisco Bay Toll Bridge Company and was opened to traffic as a toll facility on March 3, 1929. The total length of the project, including approaches to point of intersection with the nearest city or county road on either shore, was 11.63 miles. The length of the bridge structure was 7.04 miles, of which 1,503 ft consisted of steel truss spans and the remaining 6.76 miles comprised the concrete trestle portion of the structure, which is the section here under discussion. At the time of its completion this structure was reputed to be the longest overwater bridge in the world.

The concrete trestle portion of the structure consists of a total of 1,054 spans at 30 ft and 116 spans at 35 ft, the latter lying immediately adjacent to either side of the steel truss spans at the navigable channel, where the channel bottom is at a general elevation of approximately 45 ft below elevation of mean lower low water. The typical sections (Figure 1) show the type of construction for a 30-ft span, consisting of four precast concrete piles, cast-in-place cap, and precast deck sections, each consisting of two beams with long-radius circular curve soffits, and a deck slab. Two of these similar deck sections comprise the full width of the deck. The 35-ft spans are similar, except for a 5-ft increase in beam length and a 4-in. increase in beam depth. These spans also have at, 5-span intervals, two additional batter piles on each side of the cap, battered for longitudinal support in the deep water areas.

Construction procedure was to drive precast piles, followed closely by pouring cast-in-place caps. Deck sections, which were cast on shore, were barged to the site and placed on the caps, after which 10-in.-wide transverse diaphragms were poured on the caps between the ends of deck sections in the adjacent spans. These diaphragms encase the hooked ends of the main beam reinforcement and serve to tie the deck structure together longitudinally.

A transverse expansion joint through the deck section is placed at 6-span intervals in the 30-ft spans and 5-span intervals in the 35-ft spans. The two deck sections comprising each span have no transverse connection except through the caps and end diaphragms. Railing consists of posts cast in place after erection of the deck sections, with precast rail members inserted at the time of casting the posts.

At the time of purchase several of the former owner's employees skilled in the necessary shotcrete repair procedures were employed by the state, and continued in their former repair duties.

The more obvious measures required to restore the deteriorated portions of the structure as nearly as might be to their original serviceable condition, were practiced to a substantial degree by the former owner prior to purchase by the state. It should be noted that due to incompleteness of records available to the state, the dates noted for procedural changes by the former owners are approximate only, and subject to verification only through the memory of employees engaged on the work at the time.

Initially, upon determination about 1936 that a wide-spread cracking of the concrete members was developing to a serious degree, the owners commenced a program of chipping out the deteriorated portions of the concrete, cleaning the exposed reinforcement by sandblast, and replacing the concrete with shotcrete of additional thickness. This program included application of a primer and an asphaltic coating, not only to the repaired areas but to all exposed areas of piles, caps, and beams, and in some cases the bottom of deck slabs.

These corrective measures were carried out on a cost-plus contract basis until 1940, at which time the owners procured the necessary equipment and continued the repair work in substantially the same manner with their own personnel. At this time the application of the asphaltic membrane, with which a large portion of the structure had then been coated, was discontinued and the shotcreted areas were thenceforth coated with a curing agent only, as it had been determined that the areas coated with the asphaltic membrane appeared to be deteriorating more rapidly than the uncoated portions of the structure.

This program of repair was continued until about May 1942, when it was discontinued due to the war-created shortage of materials and equipment.

It was recognized by the former owners as early as 1942 that the rate of progress of the repair work was inadequate to overcome the condition of deterioration, but the war interfered with plans to expedite the work. The program was resumed in the spring of 1950, and was continued until purchase by the state in September 1951. This program has been continued in substantially the same manner by the state, which maintains a day labor crew on the shotcrete repair work.

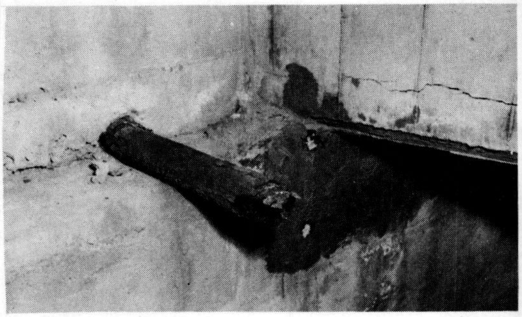

Figure 2. Typical condition of open cracks in beams. Projecting member is an iron pipe sleeve installed through end diaphragm during construction and now completely disintegrated.

However, early in the state's repair program it was recognized that the rate of progress of repairs by the day labor crew was inadequate to overcome the extensive deterioration then existing. The progress of repairs was accelerated, therefore, by letting additional repair work to contract concurrently with continued prosecution of the work by day labor. Plans and specifications were prepared, and what was considered a "pilot" contract was let for complete repairs to 48 spans of the structure, with the view to establishing the adequacy of the plans and specifications.

As this contract proceeded it became evident that the plans and specifications were adequate to satisfactorily control the quality of the resultant work, and two later contracts were let successively, each for repairs to 220 spans of the structure. Two hundred spans were actually repaired under the first of these contracts, and 220 spans will be repaired under the second, now approaching completion. Another contract is being prepared for repairs to 304 spans.

Completion of this latter contract about the middle of 1958, in conjunction with repairs effected by day labor forces, will complete repairs to all 1,170 concrete spans of the structure, by the middle of 1958. The only unrepaired deterioration remaining in the structure at that time will be progressive deterioration which has developed in the unrepaired portion of each span since the repair program was resumed in the spring of 1950. It is estimated that, failing successful development of measures to remedy the cause of further deterioration, an annual expenditure of $135,000 will be required to repair the progressive deterioration as it develops in the future.

It was recognized, as work progressed under state ownership and the character of the deterioration was closely observed upon actual removal of the fractured concrete and exposure of the corroded steel, that although the shotcrete repair procedure appeared to accomplish a satisfactory repair to the portions of the structure where it was applied, it offered no promise of arresting the deterioration in the unrepaired portions of the structure and might possibly even accelerate their deterioration. This led to the conclusion that, lacking application of other remedial measures, there was a strong probability that as a result of the process of continuous repair of newly-exposed deterioration in the unrepaired portions the structure might ultimately require complete repair in all its reinforced concrete parts, with the further possibility that the cycle of of the corrosion and develop feasible methods of arresting its advance was justified. Necessary funds were allocated and the Materials and Research Department of the Div-

of the corrosion and develop feasible methods of arresting its advance was justified. Necessary funds were allocated and the Materials and Research Department of the Di-
vision was authorized to pursue an investigation to accomplish these ends.

CHARACTER AND EXTENT OF DETERIORATION

This phase of the subject is discussed here primarily from the viewpoint of the persons charged with the duty of determining in the field the limits of deteriorated areas to be repaired, and the methods of effecting the physical repair. The cause of the deterioration is discussed in Part II.

The deterioration taking place is found to be independent of surface cracking, which might normally be considered to afford easy ingress for the corrosive effects of a marine environment. This is demonstrated when, in the course of repair, sections of concrete which on close visual inspection appear to be entirely sound, disclose the same typical spotty and discontinuous corrosion on the reinforcement as is encountered in areas where, prior to repair, the members showed continuous open cracks penetrating to the location of the reinforcement.

The pattern of corrosion as exposed does not appear to have resulted from corrosion or mill scale which might have existed on the reinforcement at the time of pouring the original concrete. The typical condition of open cracks adjacent to a beam end is shown in Figure 2. The projecting member in this view is an iron pipe sleeve installed through the end diaphragm during construction and now completely disintegrated by corrosion, demonstrating the severe effect of atmospheric exposure at the structure site.

Figure 3 shows typical cracking of a beam bottom, where longitudinal cracks generally appear on the side of the member in or near the plane of the lower main reinforcement (about 3 in. above the bottom of the member), or along the bottom surface, or in both locations. Figure 4 shows the similar condition in a cap, with the cracks lying generally in or near the plane of the lower reinforcement. Figure 5 shows typical cracking along the corner of a pile.

Figure 6 shows the spotty and discontinuous areas of corrosion in a beam bottom as disclosed upon removal of concrete. The stirrups with ends turned down are replacement parts that will later be welded into proper position where corroded stirrups have been cut away at the end of the beam. Figure 7 shows severe corrosion of ⅜-in. round stirrups at the lower bend. Figure 8 shows an area of a beam bottom where deterioration has advanced to such a degree as to completely spall off the concrete, exposing the steel.

The corrosion encountered throughout the structure is similar in type on the various structural members, but varies markedly in degree in various parts of the structure. Corrosion in the beams is generally found on the bottom layer of main reinforcement and on the lower 3 in. of the stirrups. Rarely does it become necessary to expose and clean areas on the second layer of main reinforcement, although this is sometimes necessary, generally at the beam ends.

Loss of cross-sectional area of main reinforcement is rarely sufficient to require supplementing the bar areas with additional reinforcement. However, a substantial portion of the ⅜-in. round stirrups in the deteriorated areas have suffered loss of section in the lower 3 in. sufficient to require their replacement. Occasionally corrosion is found to have progressed farther up the leg of a stirrup, in which case a chase 4 in. deep is cut along the stirrup and the repair is continued as far as necessary to properly repair the deteriorated portion.

The characteristic pattern of corrosion on the reinforcement as found in beams is irregular, discontinuous, and varies radically in degree from point to point in the

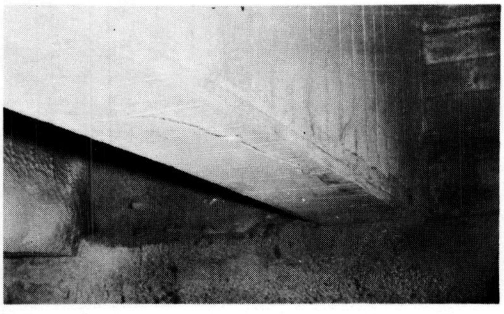

Figure 3. Typical cracks in beam. Note crack in soffit and crack in side of beam in plane of lower reinforcement.

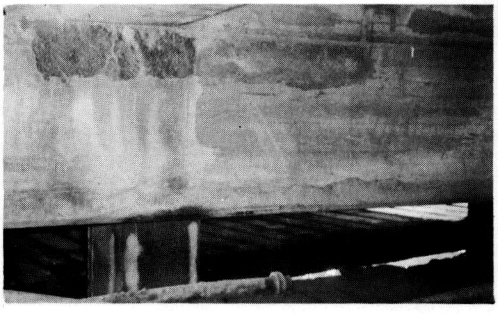

Figure 4. Typical crack in side of pile cap in plane of bottom reinforcement.

Figure 5. Typical crack along corner of pile. White areas on pile have had asphaltic coating sandblasted off to disclose possible cracks hidden by coating.

member. At a given cross-section corrosion may be heavily plated on one main bar, while the adjacent bar, less than 3 in. distant laterally, may appear to be in the same good condition as when originally encased in concrete. At a cross-section perhaps 6 in. distant from that described, the first bar may be found to be in excellent condition while the second bar is heavily plated with corrosion. In some cases severe corrosion may be found on all bottom bars at the same stirrup intersection, with the stirrup heavily corroded, while a short distance away the same transverse corrosion pattern may be seen midway between stirrups, with the main and stirrup reinforcement immediately adjacent in good condition. There appears to be no direct relation between the location of corroded areas on one main bar and similar areas on an immediately adjacent bar.

Intensity of corrosion varies from a slight trace to a ¼-in. or greater thickness of corrosion products, with little or no transverse or longitudinal relation between these varying degrees of corrosion of parallel bars. The corrosion pattern described for beams may be taken as equally characteristic of the corrosion found in piles, caps, and bottom of deck slabs. No deterioration is found in the roadway surfaces of deck slabs; open curb and railing sections show only minor deterioration, which may be classified as more or less normal deterioration in these thin reinforced concrete sections after 28 years of exposure to a marine environment.

Deterioration in caps is quite similar to that in beams, in that corrosion is found primarily on the bottom main reinforcement and on the lower few inches of the ½-in. round stirrups, with corrosion occasionally progressing farther up a stirrup leg. In deteriorated piles corrosion is generally found on main reinforcement at one or more (sometimes all four) corners. The ¼-in. round hoops in piles require replacement of the exposed portions in substantial numbers. As in beam and cap stirrups, the pile hoops in general have suffered the most severe corrosion at the bends, in many cases having completely disintegrated at these points.

The bottom surfaces of deck slabs show a relatively small degree of deterioration, but invariably this deterioration is found on those slabs which in the past were coated with asphalt. Deck slabs on large and continuous portions of the structure which were left uncoated show no evidence of deterioration, although in these areas there are occasional transverse cracks similar to normal shrinkage cracks frequently encountered in the bottom of transversely-reinforced bridge deck slabs.

The fact that these cracks have existed for many years without initiating corrosion is further evidence that the condition of deterioration found in this structure is not attributable to corrosion originating at cracks in the structural concrete. It is also notable in this regard that the occasional unfilled rock pockets encountered in the bottom of beams and caps, which may have reduced the thickness of concrete cover over the reinforcement by as much as 1 in., show no evidence of having served to initiate corrosion of the reinforcement.

As previously mentioned, various portions of the structure exhibit to varying degrees the deterioration described. The structure alignment, in general, follows a northeasterly direction for some seven miles across the southern arm of San Francisco Bay. The navigable channel, with a water depth of approximately 45 ft below mean lower low water, lies approximately 5,000 ft northeast from the west (San Mateo County) end of the structure. The structure, for approximately 750 ft to either side of the center line of the navigable channel, consists of five 300-ft truss spans on reinforced concrete piers. Between the center line of the navigable channel and the west end of the structure, the shoreward 1,500 ft shallows to the degree that the bay bottom is exposed at normal low tides. At a point about 3,000 ft east of the navigable channel the bottom shallows abruptly to an elevation about 10 ft below mean lower low water, and from this point in a distance of 5½ miles shallows gradually to the east shore. In the ½-mile of length adjacent to the east shore, the bay bottom is so shallow as to be exposed at normal low tides.

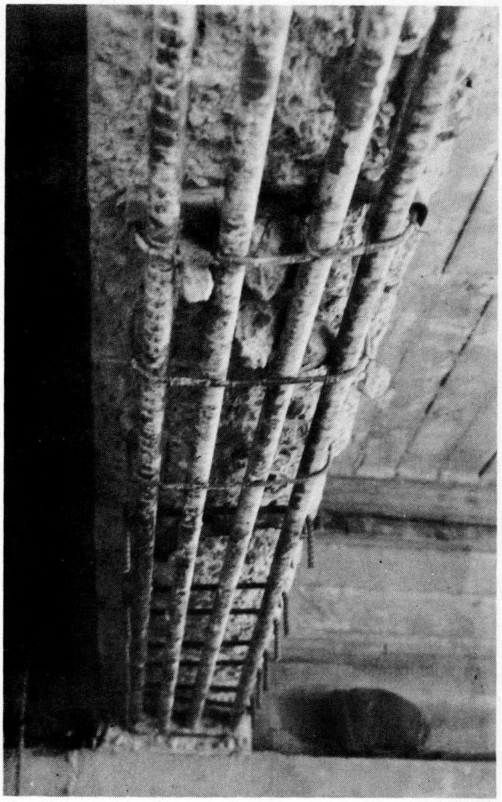

Figure 6. Beam bottom after chipping and before sandblasting, showing spotty and discontinuous areas of corrosion on reinforcement. Stirrup replacements at end of beam yet to be installed.

As shown on the typical cross-section of the structure (Fig. 1), the bottom of the caps on the trestle bents is 11.5 ft above mean lower low water, (mean lower low water = 0, mean higher high water = 5.7, extreme high tide 1956 = 10.0), and the bottom of the beams at the bearings is at Elev. 14.0. Waves of 2.5-ft half amplitude are of frequent occurrence. These structure elevations are practically constant throughout the length of the structure except for the 4-span ramped sections at each end of the structure where these elevations decrease by 1.75 ft, and at the 7-span ramped sections adjacent to either end of the steel truss spans at the navigable channel, where these elevations increase a maximum of 7.3 ft.

The general degree of deterioration is more or less uniform from the west shore to a point 4.8 miles therefrom. In this portion of the structure piles, caps and beams were, with relatively minor exceptions, coated with the aforementioned asphaltic coating, and the bottom of deck slabs in the west 1.6 miles of this portion were, with minor exceptions, similarly coated. From the point 4.8 miles from the west shore to the end of the structure at the east shore (a length of 2.2 miles) the piles only were coated with the asphaltic coating. In this portion of the structure the degree of deterioration in beams and caps and piles gradually decreases in intensity until, at the easterly end of the structure, the degree of deterioration of beams is approximately 10 percent, of caps about 50 percent, and of piles about 70 percent of the degree of deterioration found in the more seriously deteriorated westerly 4.8 miles of structure length. It is notable that in the easterly 2.2 miles of the structure only the piles were coated with an asphaltic coating, and, as previously noted, it is this structural member which shows the degree of deterioration more nearly comparable to that found in the more seriously deteriorated 4.8-mile westerly portion of the structure.

The six piers supporting the steel truss spans are supported on precast concrete piles, and consist of two conical shafts joined by a transverse web wall and a cap slab. The top of the 2-ft-thick cap varies from Elev. 20.0 to 39.0 on the various piers. The

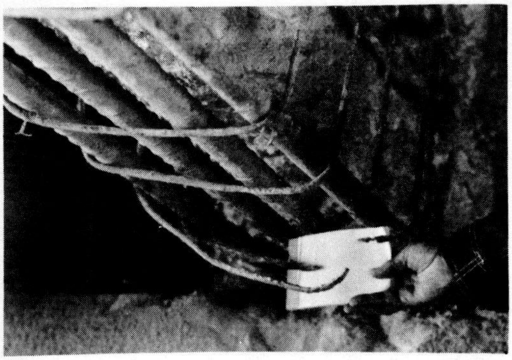

Figure 7. Tapered ends of severed 3/8-in. round stirrups show almost 100 percent loss of cross-section.

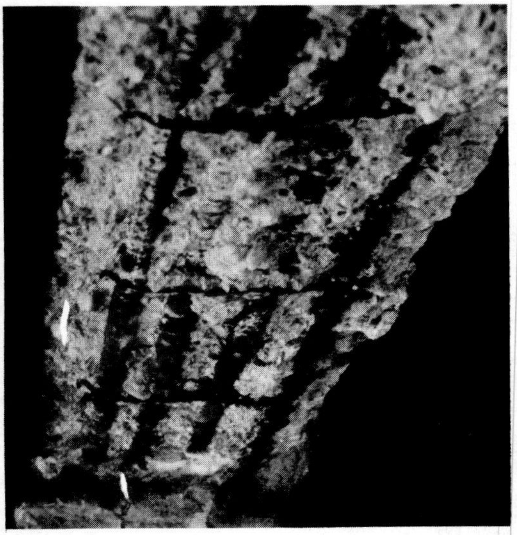

Figure 8. Area where deterioration has advanced to such degree as to spall off a considerable portion of the beam bottom.

pier surfaces were treated with the asphaltic coating. Although the upper portions of the exposed surfaces of these piers are substantially higher above the splash and spray zone than the concrete trestle structural members in general, the pier surfaces above water have suffered the typical deterioration to a degree which in the past necessitated shotcrete repairs over considerable portions of their surfaces, these repairs extending as high as the bottom of the pier caps. These notable variations in the degree of deterioration appear to indicate that those portions of the structure that were treated with an asphaltic coating have beyond a doubt suffered the most severe deterioration.

A further factor, which is under investigation as a possible explanation of the lesser degree of deterioration encountered in the asphalt-coated piles in the easterly 2.2 miles of the structure, where the degree of deterioration of this member as previously noted is found to be approximately 70 percent of that found in other portions of the structure, is the indication that the relative atmospheric humidity decreases as the easterly shore is approached.

BASIS OF PAYMENT FOR REPAIRS

During the period when repairs were effected by cost-plus contract, or directly by owner's personnel, there had been no necessity for establishing a quantity basis for payment. However, when the state decided to contract the repair work it became necessary to provide for payment on a unit price basis. The commonly specified bases of payment for shotcrete repairs (namely, payment on surface area repaired or payment on volume of shotcrete placed) were considered. However, long experience in repairing this particular structure had indicated that the portion of the cross-section of the various types of members to be chipped away, and consequently, the cross-sectional area of shotcrete to be replaced for a satisfactory repair, could be dimensionally standardized.

For simplicity of determining in the field the quantities for payment, it was decided to standardize the cross-sectional demensions of the repair and to use the length of the repair as a basis of payment, to the extent practicable. This results, for most pay items, in reducing the field quantity survey for payment to a simple measurement of length of each member chipped away. An item is included on an area basis for payment for repair of random areas on sides of members. All random area repairs are chipped a standard 4 in. deep.

On the previously mentioned pending contract for repairs to 304 spans it has been found necessary to include in this item required repairs to the bottom of the deck slab. This previously was not considered necessary, as all portions of the structure repaired by unit price contract were portions where the bottom of the deck slab had not been treated with the asphaltic coating except for one short section of 12 spans, and where no

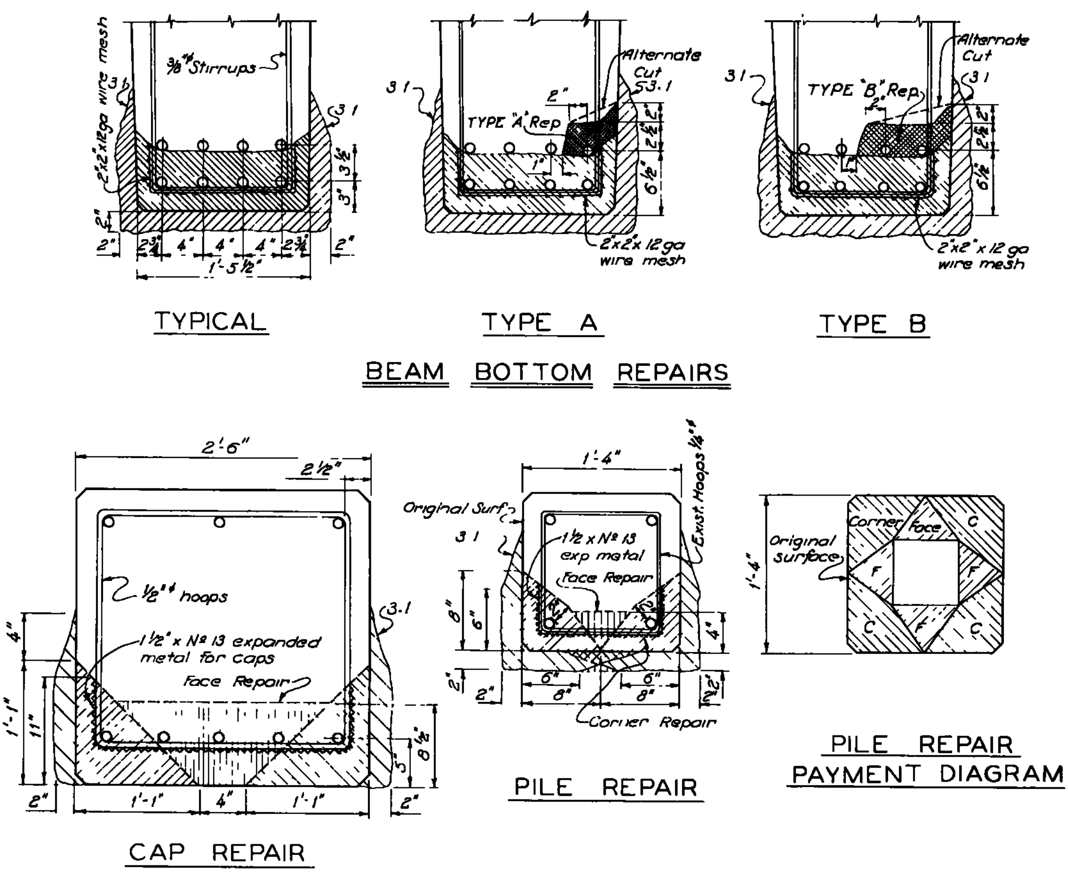

Figure 9. Typical cuts and details shown on contract plans.

deck slab repairs were required. An item is included for removal of previously installed shotcrete jackets, which completely encased four sides of many piles, and this item is paid for on a surface area basis. These encasements have proven to be a handicap in cases where less than all four corners and four faces of a pile were chipped and repaired, as they have served to obscure the continued development of deterioration on the unrepaired parts of the pile so encased, and must be removed when further repairs are required in the jacketed area. The present repair policy eliminates replacement of encasements over unrepaired portions of piles, except in the case of serious damage to the original pile surface, which might occur during stripping of the formerly installed shotcrete jacket.

Payment for the various items of repair between Elev. +3.5 and -0.5 is segregated from payment for work on the same items above this elevation, on the theory that work below this level requires a closer coordination with daily tidal range, and may involve crew overtime or delays in moving staging forward, pending a period of low tide of sufficient duration to complete the lowest portions of the repair. Only infrequently is it found necessary to carry repairs as low as Elev. -0.5, and it is these repairs which require extremely close coordination with the tidal range. No repairs have required below Elev. -0.5, except at the shallows on the westerly end of the structure where repairs have been carried below the exposed channel bottom during periods of low tide. It is fortunate that deterioration has rarely progressed lower than the elevation of mean lower low water, which makes it possible to carry out pile repairs in the open without the need of cofferdams.

Because repairs may be required on parts of the cross-sectional area of a member only, such as one corner only, or two corners and a face of a pile or cap, or three cor-

San Mateo-Hayward Bridge

SURVEY OF DETERIORATED CONCRETE

SPAN NO.	BEAM NO.	CAP NO.	PILE NO.	TYPE OF REPAIR	LOCATION	LENGTH OF REPAIR	AREA OF REPAIR	R E M A R K S
X	N1			B.B.		16 ft		
	N2			"		14		
	S2			"		27.5		R.C. 5 ft
	S1			OK				
		X		1C		36		R.P. - 40 sq ft
				2C1F		10		
			N1	2C1F		10		R.J.-10 ft (2' below + 3.5)
			N2	OK				
			S2	2C1F		6		R.J. 6 ft
			S1	3C2F		6		
				Deck Slab			20 sq ft	

Note: B.B. indicates typical beam bottom repair.
R.C. indicates repair 5 linear feet of secondary crack to be sealed.
R.P. indicates 40 sq feet of previously placed shotcrete to be removed. (recorded for purpose of payment for removing a ditional thickness beyond the original cross-section)
R.J. indicates previously installed shotcrete jacket to be removed.

For estimating purposes no record was made of the exact location on the member of the portion to be repaired. This is recorded later during actual repair.

Figure 10. Field survey record sheet.

ners and two faces of a pile, the basis of payment for such repairs is established as the length of single corner or single face repaired. The basis of what might be called the "unit-cut" method of payment is shown for the various types of members in Figure 9. The pile repair detail shows the repair for two corners. Similar details apply for any number of corners or faces. The payment diagram for pile repair indicates the concrete to be removed for any permissible combination of corners and faces. The cap repair detail shows a repair of two corners and one face. Similar dimensions apply also to repair of one corner only. The typical beam bottom repair detail applies to all beam bottoms requiring repair, whereas the Type A and Type B beam bottom repairs are superimposed on the typical repair to provide for repair of one to four bars in the second layer of beam reinforcement. On this basis of payment, if a pile is repaired around its complete circumference for a length of 2 ft along its axis the Contractor receives payment for four corners at 2 lin ft each, and four faces at 2 lin ft each, or a total of 8 lin ft of pile corner repair and 8 lin ft of pile face repair, at a unit price bid for each of these two types of repair. As the corner cuts and face cuts are defined on the plans it is impractical to remove a face without removal of both adjacent corners, as an undercut would be required at the side surface of the face cut against an unremoved corner. It is therefore specified that where a face is to be removed both adjacent corners will be removed.

In the case of beam bottom repairs the whole bottom is always removed to expose the lower layer of main reinforcement, and the length of this repair is paid for at the unit price bid for beam bottom repair. Additional pay items for beam bottom repair, Types A and B, are included to provide for the relatively infrequent need to expose bars in the second lowest layer of beam reinforcement, and are paid for on a linear foot basis in addition to the payment made for the standard beam bottom repair. In accordance with Division policy, the steel items incorporated in the work are segregated and the quantities of each of these items used is paid for in place, on a unit price basis. These items are bar reinforcing steel, paid on a per pound basis, and wire mesh reinforcement and expanded metal lath reinforcement, paid on a square yard basis.

The item of traffic control is paid for on a lump sum basis, and includes 24-hr daily manual control of an electrically-operated system of 3-color signal heads and advance warning amber flashers, furnished by the state but operated by the contractor. Signal heads and amber flashers (a total of four each) are located on each side of the 27-ft roadway at each end of the work area. Suitable warning signs are furnished by the state but maintained by the contractor. A work area occupying one-half of the roadway width for a length of 510 ft (17 spans at 30 ft) is allowed for the contractor's operations. The work area is kept as short as is consistent with adequate working room, to reduce to a minimum the time required for vehicles to clear the one-way traffic zone when signals indicate a reversal of direction of traffic flow.

Currently the contractor scaffolds 12 spans at one time. The one-half width of the roadway under repair at any one time is closed off by barricades. The whole system of control signals, signs, barricades, and work equipment is moved forward periodically as the work progresses. All obstructions are removed from the bridge deck from Saturday morning to beginning of work on Monday morning, thus leaving the roadway clear for the heavy week-end traffic. During this period the signal system is not operated, and all warning signs and signal heads are covered or turned normal to the center line of roadway so as to have no significance for passing traffic.

Although the work under way might otherwise lend itself to performance from barges, with the consequent advantage of complete elimination of all obstructions from the roadway at all times, the fact that a substantial portion, and in many cases the full length, of the beam bottom is chipped away and the member is thus materially weakened, dictates that traffic not be allowed to use the portion of the roadway under repair for a minimum of 36 hr after beam repairs are completed. This makes it mandatory that

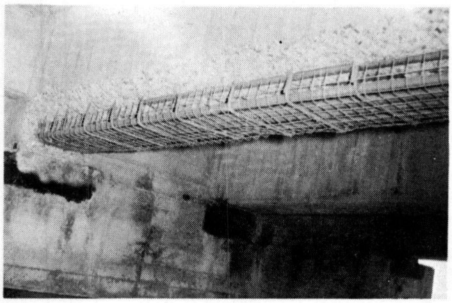

Figure 11. Beam bottom with sturrups replaced and wire mesh in place, ready for shotcrete.

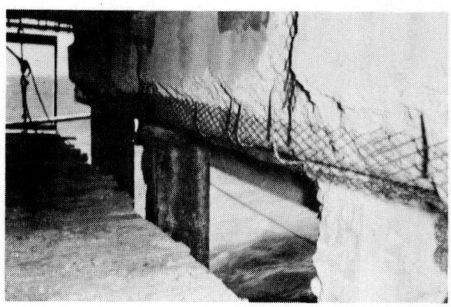

Figure 12. Cap corner ready for shotcrete. Note vertical chases cut to remedy corrosion on stirrup legs.

the contractor complete all beam repairs by the end of the work period on Thursday in order that beam repairs may be 36 hr old by Saturday morning, at which time the full roadway width must be made available for use by traffic.

The time required before beams may be allowed to carry the live and impact loads of moving traffic explains the choice of high early strength portland cement for use in this work. The fact that the superstructure was originally installed in two independent half-width sections is of great benefit to the repair operation, as deflections and vibrations in the portion of the roadway carrying traffic are not transmitted to the half of the superstructure under repair, and are negligible in the substructure members.

ESTIMATING PROCEDURE TO ESTABLISH CONTRACT QUANTITIES

As previously mentioned the decision to perform repairs on the unit price contract basis posed certain problems, foremost of which was pre-determination of the quantity of work anticipated for each contract item. The decision to adopt the unit-cut basis of payment (that is, separate payment for individual corners and individual faces of members) was of primary importance in simplifying the cataloging of the various contract item quantities during the field inspection. On this basis it was only necessary to inspect for the length of repair and the number of corners and faces involved in the repair of the particular member. Thus, if visual inspection of a pile indicated three corners and two faces to require repair for a length of 5 lin ft, this was recorded in the field as: pile, three corners, two faces, length 5 ft, with the added notation of how much of this length was below Elev. +3.5. This was later reduced during quantity tabulation as: repair pile, one corner, 15 lin ft, and repair pile, one face, 10 lin ft, with the further segregation of the portions of this tabular quantity which were above or below Elev.+3.5.

Field recording of cap repairs required only determination of the number of corners and faces requiring repair and the length of the repair. It was necessary during inspection of both piles and caps to take continuous cognizance of the fact that where a face was to be removed both adjacent corners would be removed. Thus, if inspection of a cap from one side indicated 28 lin ft (full length) of one corner, and the end 8 ft of the bottom face required repair, while inspection from the opposite side indicated that the second corner required repair for a 15-ft length from the end where the bottom face was sound, this was recorded in the field as: cap, two corners and one face, 8 lin ft, one corner (20 ft plus 15 ft) 35 lin ft, which was later reduced to the final contract item figure of: cap, one corner, 51 lin ft; cap, one face, 8 lin ft.

A field inspection record sheet (Fig. 10) is prepared for each span of the structure. Inspection is carried out from a flat-bottomed work boat about 6 ft wide by 20 ft long, with a flat working deck about 6 ft wide by 12 ft long. This craft is fitted with an outboard motor for propulsion to the general site of the work. It has a freeboard of about 1 ft and a draft of about 8 in., and is thus capable of entry into the shoreward shallows. A crew of two boatmen, a recorder, and two inspectors is used.

After arrival at the inspection area the boat is pushed about each span and from bent to bent by the two boatmen using boat hooks, as directed by the inspectors. Because this boat is not adaptable to rough water, and since this area of San Francisco Bay is generally quite choppy in the afternoon during the most suitable season for carrying out inspection work, it is generally necessary to cease inspection work about 1:30 PM and return to dock. The work is begun about 8:00 AM. Piles are inspected during the portion of this available work period when the tide is lowest. Piles in each bent are inspected from both sides as the boat makes a pass along one side of the bent and returns along the opposite side. The boat always lies parallel with the bents, and when moving from bent to bent is pushed sidewise across the span. This affords the inspectors the best viewing opportunity.

Each inspector observes a contiguous pair of the four piles, observing visible signs of cracks, rust stain, and bulges in the asphaltic coating, which indicate water pockets under the coating and may indicate cracks in the concrete. A prospector's pick is used to remove the asphaltic coating in doubtful areas where cracks are suspected beneath th coating, and for sounding the concrete; flat scrapers are used to remove the barnacle growth where closer inspection is desirable. The inspector calls to the recorder the name and number of the member and the type and length of repair his judgment dictates is required.

This notation may be added to when the pile is inspected from the opposite side; that is, a repair which may have been classified as two corners and one face when viewed from one side only may later be classified as three corners and two faces when the oppo site side is viewed. The fact that large areas of the structure are coated with the asphaltic coating, which bridges over and hides many of the tighter cracks from view, makes it difficult to arrive at an accurate determination of the actual extent of surface cracking, which is the basis of the survey. However, it is not considered practicable to apply sandblast to remove portions of the asphalt coating in suspect areas during inspection, and successful execution of this type of inspection hinges upon close visual inspection combined with experience gained in observing members for cracks while the asphaltic coating is actually being partially removed during the progress of repairs. Length of anticipated repair is estimated liberally durin the progress of repairs. Length of anti-

Figure 13. Contractor's equipment for batching, mixing, and shooting shotcrete.

cipated repair is estimated liberally during this inspection for two reasons: first, repair experience dictates that the actual repair will probably extend about 2 ft past the closed end of a visible crack on an uncoated member; and second, many very narrow cracks are not visually detectable through the asphaltic coating. However, a large percentage of cracks are visible through the asphaltic coating, due to the slight surface offset which may be present during the spalling process, and to stretching of the asphaltic film across the crack, which becomes apparent when light strikes this area from a favorable direction.

Figure 14. Showing outboard end of Contractor's staging. Each stage spans more than full width of structure deck.

As the tide rises during the progress of the inspection and the overhead beams come closer to the view, inspection of piles is discontinued and inspection of beams is begun, perhaps in the same group of spans just passed through for inspection of piles. Beams are similarly inspected in contiguous pairs. In order to have a close view of the beam under inspection, the boat again makes two passes through each span. On one pass one-half the length of the four beams is inspected (from one bent to center of span). The boat is then moved the length of the span to the next bent and returns through the same span along the bent. This procedure affords a close view of the beams at the supports and a second view of the central portion as the boat passes across the span.

Upon completion of the field survey quantities are tabulated on the basis of contract unit items. At this stage proposed contract quantities for each item are adjusted on the basis of comparison of past field surveys with quantities of the various items actually repaired on completed contracts. This comparison serves to directly relate the quantities visually determinable in the field to the quantities actually disclosed under close-up inspection during progress of repair work. Development of a reasonably accurate estimate of quantities of work to be done on the first "pilot" contract was especially difficult, as there were at that time no data available for a comparison of quantities established by field survey with quantities actually repaired. However, past contract experience now available as a check is of material assistance in development of realistic quantities of work to be performed. Anticipated quantities of certain items of repair which are not susceptible of visual determination in the field, such as beam bottom repairs, Type A and Type B, and quantities of bar reinforcement which will require replacement, are estimated on the basis of quantities required in past repair work.

REPAIR PROCEDURES

The specified methods of shotcrete repair are based on the American Concrete Institute standard of "Recommended Practice for The Application of Mortar by Pneumatic Pressure" (ACI 805-51), with minor variations appropriate to the particular work under way. Figures 11 to 15 inclusive show various details of the work in progress.

This discussion primarily concerns itself with the deviations from the recommended practice and the reasons therefor.

Shotcrete ingredients consist of high early strength portland cement, a specially-blended shotcrete sand, a pozzolanic additive, and water from a potable supply. High early strength cement is used, rather than the normally specified Type II, for convenience of traffic. Sand consists of a specially-blended sand composed of a normal washed concrete sand meeting the Division's grading specifications for fine aggregate for Class "A" (6 sacks of cement per cubic yard) concrete, to which is added approximately 12 percent of a fine gravel, resulting in a sand grading falling within the following specified limits:

Sieve Size	Passing Sieve, %
3/8 in.	100
No. 4	95 - 100
No. 8	70 - 80
No. 16	40 - 50
No. 30	20 - 30
No. 50	7 - 15
No. 100	2 - 5
No. 200	0 - 2

The coarser gravel aggregate addition to the concrete sand has been found to be highly beneficial in eliminating rebound material during shotcreting around and behind closely placed reinforcement. The mix specified is 1: 3 by dry weight, with a pozzolanic additive of not to exceed 5 percent by weight of cement. Strength is specified as 3,500 psi at 7 days and 6,000 psi at 28 days. These strengths are readily obtainable with the specified mix, and in practice 28-day strength ranges as high as 9,000 psi. Mixing time is specified at 1.5 min for drum or pubmill mixers.

Currently the contractor uses a continuous-type mixer, which delivers an intimately-mixed and uniform product apparently equal in all respects to the product produced by the pubmill-type mixer in use by the state's day labor forces. Nozzle velocities, nozzle pressures (air and water), increase of nozzle pressures with excessive hose lengths, wetting contact surfaces prior to shotcreting, preventing inclusion of rebound material, cleaning surface of first course prior to placing second course, curing the finished work, and other normally required shotcrete procedures are specified in accordance with the previously mentioned "Recommended Practice."

Prior to commencing repair of a member the contractor is required to remove portions of the asphaltic coating by sandblasting a zig-zag line along all faces of the member, for purposes of locating hidden cracks. After this is done the inspector marks the length for repair. Extreme care is used to assure a removal of all corrosion products from the reinforcement. For this reason a minimum of 2 in. of concrete is removed behind reinforcement requiring sandblasting in order to facilitate through cleaning of the back side of the reinforcement. Existing reinforcement is thoroughly sandblasted prior to installing additional bar, mesh, or expanded metal lath reinforcement.

The prepared member is again lightly blasted shortly prior to placing shotcrete. This serves to clean all welds and to remove any light film of rust which may exist on the added reinforcement, or may have developed overnight on previously cleaned reinforcement. Expanded metal lath is used on caps and piles instead of the normally used wire mesh due to its greater stiffness, which is of material benefit where support for the mesh provided by relatively closely-spaced stirrups and bar reinforcement is lacking. Rebound is less readily removed from behind the expanded metal lath than from areas covered with wire mesh. For this reason it is not considered suitable for

Figure 15. Nozzleman "shooting" beam requiring repair through part of its length.

use on beam bottoms where reinforcement is closely placed and clearances behind exposed reinforcement are small. Repair of piles in the tidal range is specified to be completed a minimum of 1 hr prior to submersion by a rising tide, which interval is adequate to prevent scour at this location.

Shotcrete is applied in not less than two courses, due to the depths of material required to be placed. In general, the first course is applied to cover all reinforcement and the second course conforms to the final dimension desired. Special care is required on the overhead work involved in shotcreting bottoms of members, in order to avoid the tendency of excessive thicknesses of shotcrete applied in one course overhead to slough off.

In practice it is found to result in better work in shotcreting beam bottoms where the most critical part of the work (that is, filling in behind and around the closely-placed main reinforcement) is done overhead, if the nozzle is held approximately 1 ft from the work rather than at the more usual 2½- or 3-ft distance. This serves to more effectively remove the rebound material, which at this location tends to fall and deposit on the upper surface of the reinforcement, with the consequent possibility of inclusion in the finished work.

The work equipment and materials are located on the bridge deck, where they are confined to one-half the width of the 27-ft roadway, leaving one 13½-ft lane for controlled one-way traffic. Mixing of shotcrete ingredients is done on the bridge deck, where the shotcrete gun is located.

The actual work of concrete cutting, sandblasting, shotcreting, etc., is carried out from timber stagings suspended below the bridge deck and controlled by separate manually-powered hoist units. The contractor's preference is to scaffold each span with two large stages (12- by 36-ft in size), which provide practically a complete working deck underneath the span. Each of these units is controlled by four ¾-ton ratchet hoists, one at each corner. All stages are moved from span to span by lowering until they are afloat, moving the hoists ahead and towing the stage to the new location by men walking along the bridge deck.

No operating equipment (such as air compressors) capable of creating an appreciable vibration in the superstructure is allowed to operate on the bridge deck in a span where beams are being shotcreted or have been shotcreted within 36 hr, as such vibration tends to dislodge newly-placed shotcrete, particularly from an overhead location. In this respect, no appreciable vibration is transmitted by passing traffic to the area of deck supported by beams which are under repair as a result of the original method of construction, which was to cast and place the superstructure in two independent halves joined transversely only at their ends.

The contractor on the current work uses approximately 215 sacks of cement daily, and estimates that the resulting mix provides approximately 9¾ cu yd of shotcrete in place in the structure. This completes repairs in the equivalent of 1¼ 30-ft spans daily. The work is done with an average crew of 33 men, including three flagmen. One Size N-2 gun is in use on the work.

REPAIR COSTS

The cost of shotcrete repairs accomplished during the period from September 12, 1951 (date of purchase by state) to September 30, 1956, and the anticipated costs of work to be done during the period October 1, 1956 to June 30, 1958, at which time it is anticipated that all reinforced concrete spans in the structure, except the equivalent of 50 spans, will have been fully repaired by procedures under state control, are given tabulated below. The equivalent of 50 spands done prior to September 12, 1951, consists of partial reparis accomplished by the former owners in a group of 132 spans, from resumption of the repair program in the spring of 1950 until purchase by the state. This work is excepted from the number of spans to be repaired after purchase by the state, as it remains perfectly adequate at this time. These costs include construction engineering charges in all cases.

The total number of spans repaired and yet to be repaired by state day labor forces are approximate only, as work was partially done by these forces on certain spans in

TABLE 1

COSTS OF REPAIRS TO CONCRETE STRUCTURE OF SAN MATEO-HAYWARD BRIDGE

Type of Operation	Dates	No. of Spans	Cost, $	
			Total	Per Span
Day labor	9/12/51 - 9/30/56	263	518,626	2,200
Contract 1	to 9/30/56	48	130,520	2,719
Contract 2	to 9/30/56	200	547,769	2,734
Contract 3[a]	to 9/30/56	183	373,206	2,039
Day labor[b]	to 9/30/58	85	187,000	2,200
Contract 3[b]	to 9/30/58	37	66,794	1,805
Contract 4[b]	to 9/30/58	304	574,000	1,888
Total		1,120	2,457,915	2,195

[a] Only partially complete.

[b] Estimate on anticipated work.

which the former owners had made partial repairs, as previously noted. During final negotiations for purchase it was agreed that the former owners would continue the repair work and that the state would reimburse them for the cost of said work. This agreement was in effect from May 1951 to September 12, 1951, during which time $23,911 was expended by the former owners for repair work, for which they were reimbursed by the state in addition to the agreed purchase price. The spans partially repaired under this agreement, and the cost of these repairs, are not included in the cost tabulation (Tables 1 and 2) for work done after purchase by the state.

Costs for work under the current contract to September 30, 1956, are approximate only, as contract payments earned between September 20, 1956 (latest date covered by contract estimate), and September 30, 1956 have been estimated, and costs of construction engineering are incomplete.

It should be noted that the cost of repair per span decreases toward the easterly end of the structure where the degree of deterioration has been described as decreasingly severe. All work done to date and yet to be done by day labor forces will have been carried out in the westerly half of the structure length where the degree of deterioration is considered normally severe. Work under Contract 3 lies in the westerly portion of the easterly 2.2 miles of structure length, where the deterioration begins to decrease in severity, and it is noted that the first 183 spans of this 220-span contract have been completed at a cost of $2,039 per span, whereas it is estimated that the remaining 37 spans of this contract will be completed at a cost of $1,805 per span. The proposed contract for 304 spans will include the easterly 176 spans of the structure, where the deterioration is least severe, plus a section of 128 spans in the westerly half of the

TABLE 2

COMPARATIVE COST FOR REPAIRS BY DAY LABOR AND BY CONTRACT; SAN MATEO-HAYWARD BRIDGE

Type of Operation	No. of Spans	Cost, $	
		Total	Per Span
Day labor	348	765,626	2,200
Contract	772	1,692,289	2,192

structure where the deterioration is considered to be normally severe. This combination in the proposed Contract 4, of a portion of the structure where deterioration is severe, combined with a portion of the structure where the deterioration is least severe accounts for the relatively low estimated average cost of $1,888 per span.

CONCLUSION

This discussion illustrates the magnitude of the problem presented by deterioration of the reinforced concrete in the structure described. The actual physical repair of the structure, although costly due to the unusual length of the structure, presents no unusual problems. However, because many other reinforced concrete bridges in the San Francisco Bay area and in other parts of California have successfully withstood exposure to the marine environment for periods of 30 years or more, the causes of this serious structural deterioration, which had made itself evident as early as seven years after completion of construction, must be sought for in some unfortunate combination of circumstances applying to this structure in particular. It is hoped that the investigation now underway by the Materials and Research Department of the Highway Division will disclose evidence of the basic causes of the condition and suggest appropriate methods of arresting its advance.

The characteristic pattern of corrosion effects has been described in considerable detail, with the thought that it may be helpful in identifying similar conditions elsewhere and correlating such visual evidence with the basic cause, discussed in Part II. It would appear that when a similar corrosion pattern is evident more than the simple physical repair is desirable, and that further investigation to determine the basic cause is justified. It is further indicated that, under similar conditions, it is unwise to partially repair the perimeter of a member where in the course of such repair the unrepaired portion of the perimeter is also encased with a shotcrete jacket, as the jacket on unrepaired portions serves to hide the effects of further deterioration and increases the cost of later repairs.

The unit-cut basis of payment for shotcrete repairs has been found to provide accurate quantity control in all phases of the work, with a minimum of effort in both office and field. In this connection it should be borne in mind that this basis of payment is most readily adaptable to situations where the primary requirement of the repair process is that the reinforcement be properly exposed for thorough elimination of corrosion products from the reinforcement. Had the primary problem been the removal of defective or deteriorated concrete, rather than exposure of reinforcement, it is probable that payment on the basis of length of repair might not have been so conveniently or accurately applied.

The average annual cost of repairs to this structure throughout its service life, due to repair of deteriorated reinforced concrete, has been comparatively high. No cost figures are at hand for work done by the former owners from 1936 to 1942 and from 1950 to 1951. However, expenditures for this purpose by the state alone, from September 1951 to June 1958, will approximate $2,458,000. No costs of other maintenance work which might be considered normal for such a structure are included in this figure. Because the entire structure was constructed at a guaranteed contract price of not to exceed $5,000,000 some 28 years ago, this indicates that two years hence, after 30 years of service life, the state alone will have expended 49 percent of the original cost of construction, which is an average annual expenditure of 1.64 percent of original cost throughout the life of the structure.

As previously discussed, failing successful application of measures to arrest the progress of the deterioration, it is estimated that a continuing annual expenditure of $135,000 will be required to keep the progressive deterioration repaired. This annual sum is 2.7 percent of the original cost. It is hoped that the final results of the investigation discussed in Part II will disclose a practicable means of materially reducing this estimated future annual expenditure.

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 responsisible 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	New Shotcrete[a] 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

[a]V = 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 soniscope 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 ¼-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	-	-	-

[a]Drop in air pressure in a closed container connected to one face of specimen, the other face being at atmospheric pressure.

[b]2-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 $Ca(OH)_2$. 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	New Shotcrete[b] 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					
	5.0	5.4	1.1	0.0	0.4	1.3
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					
	3.5	4.0	1.1	0.0	0.2	1.0

[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

[a]Position 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: E $\times 10^{-6}$	Beam Cores: Weight, pcf	Beam Cores: Absorption, %	Beam Cores: MgO, %[b]	Beam Cores: Cl, %[c]	Pulse Vel., fps	Resist. in Anodic Areas, ohm-cm
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

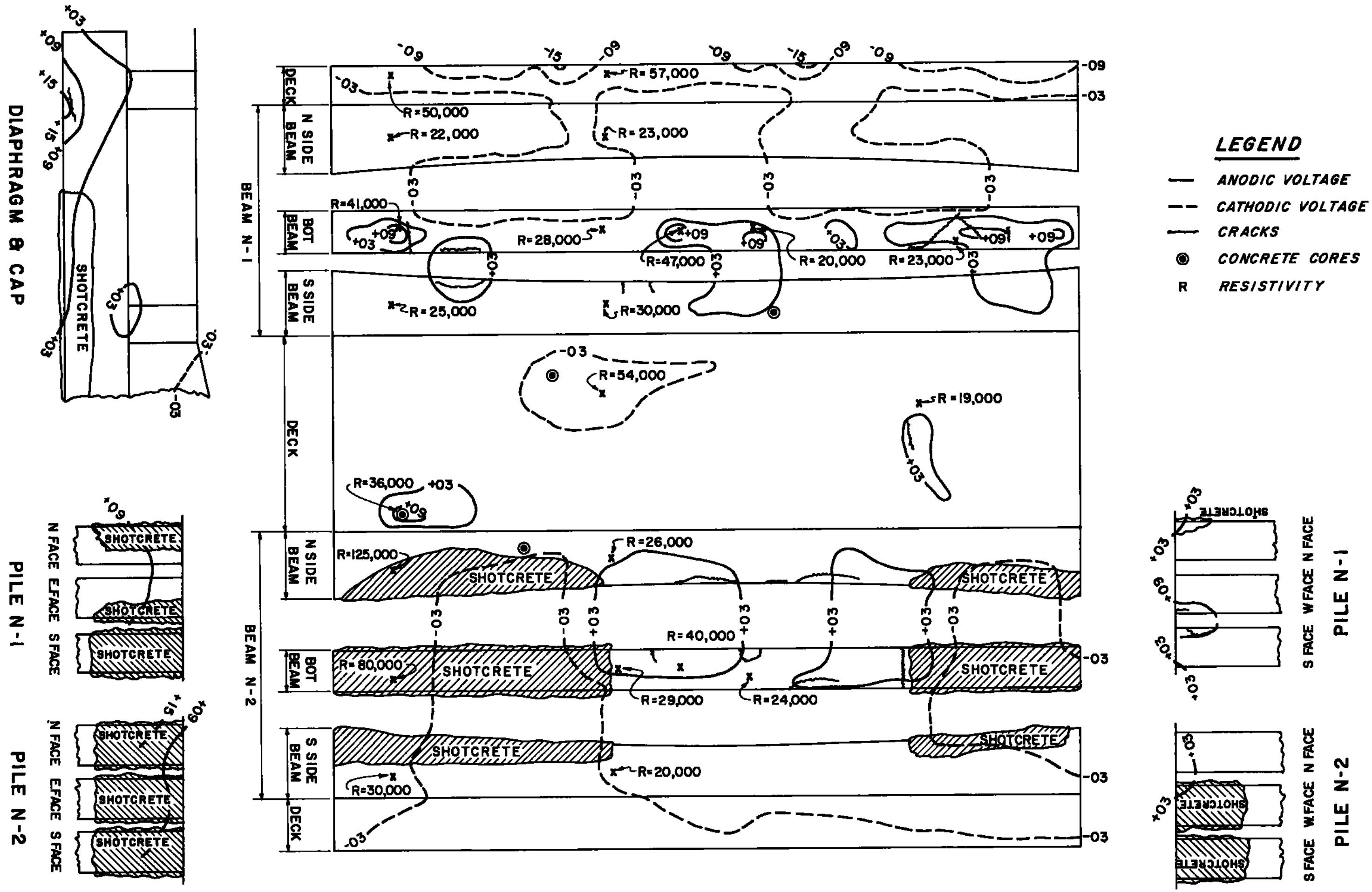

Figure 16. Equipotential contours, Span 70.

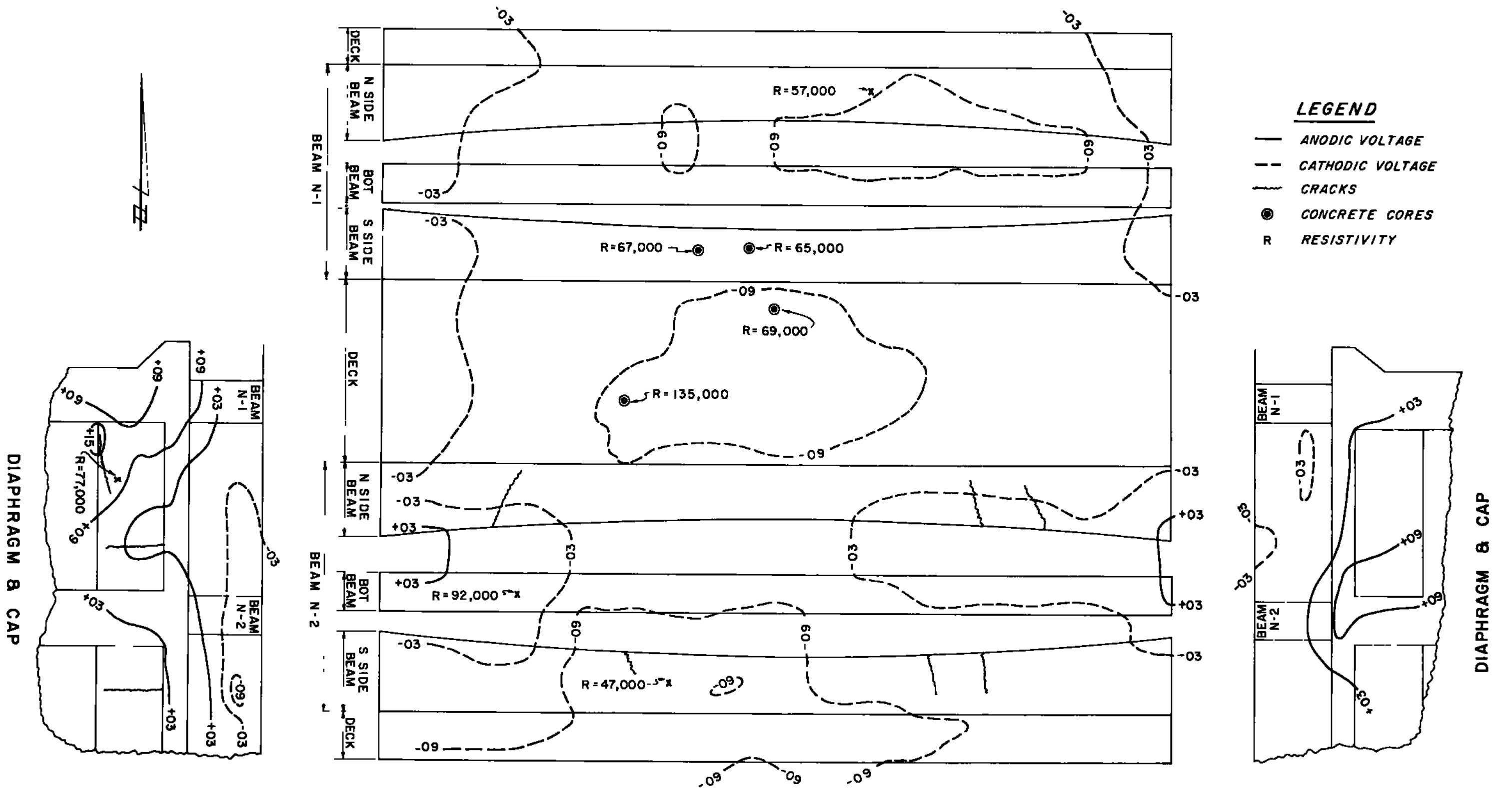

Figure 17. Equipotential contours, Span 163.

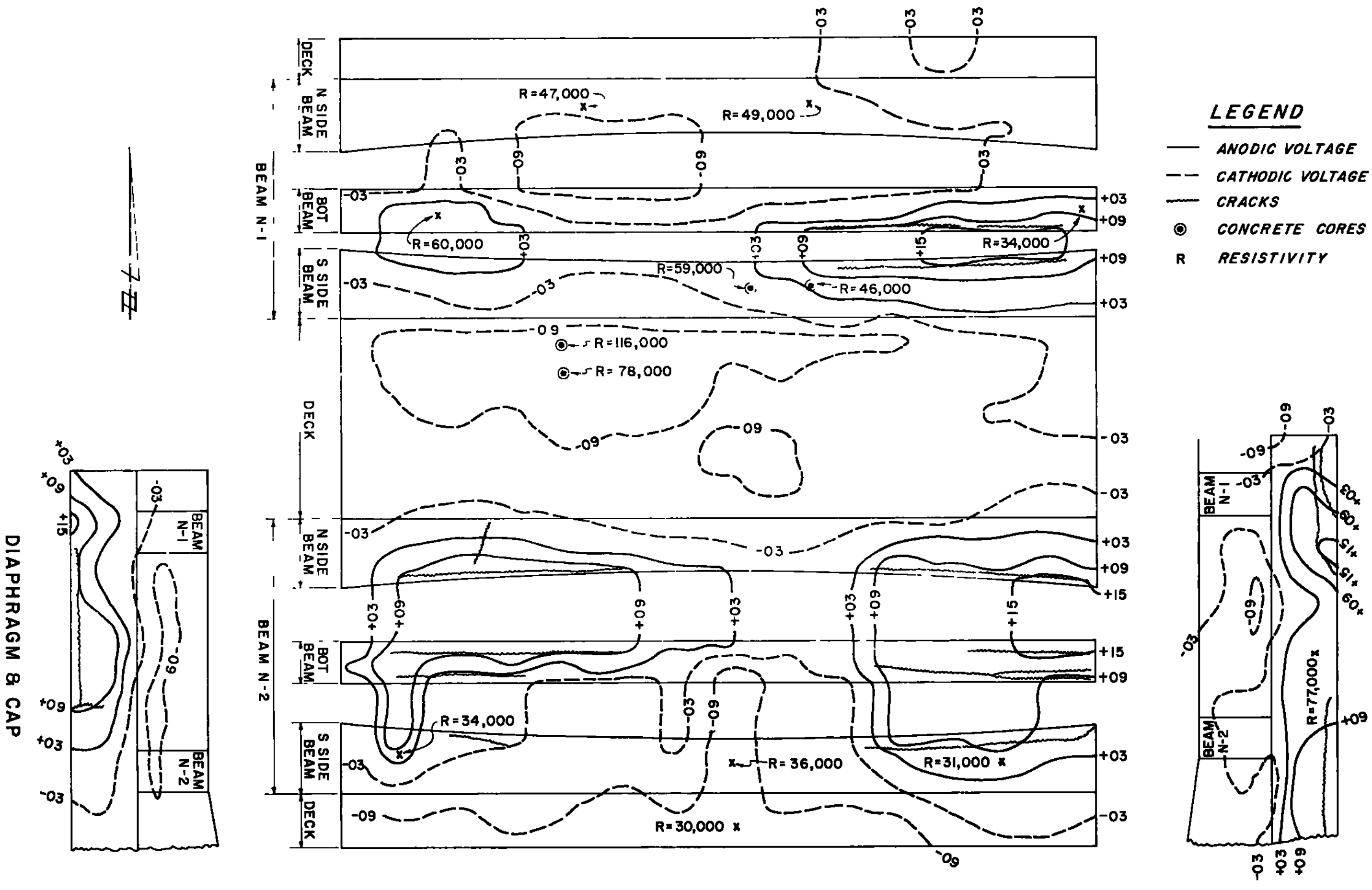

Figure 18. Equipotential contours, Span 553.

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

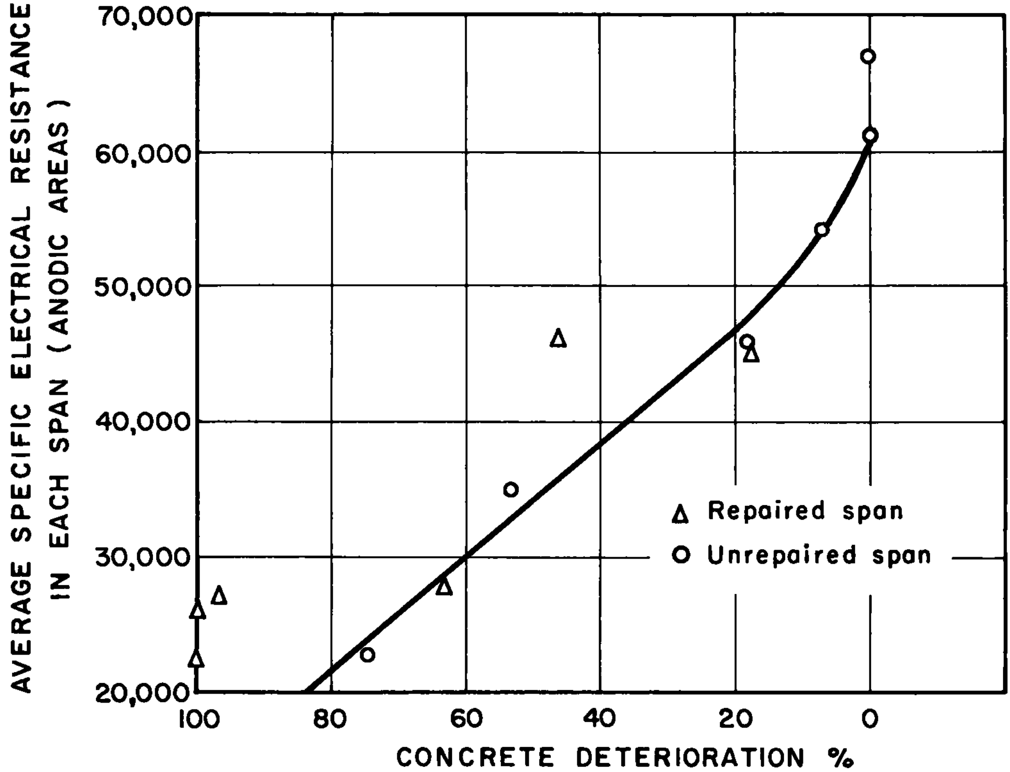

Figure 19. Specific electrical resistance against concrete deterioration.

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

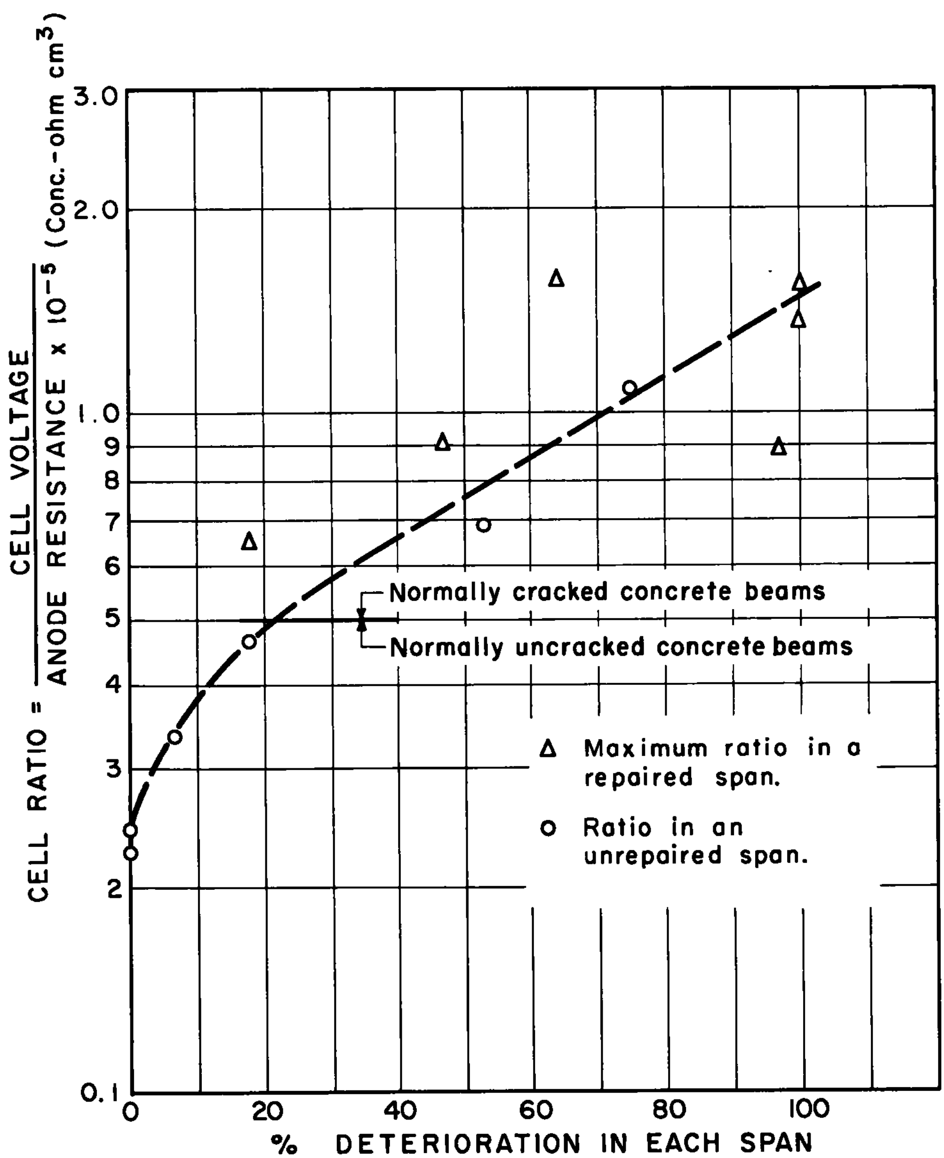

Figure 20. Corrosion cell ratio against concrete deterioration.

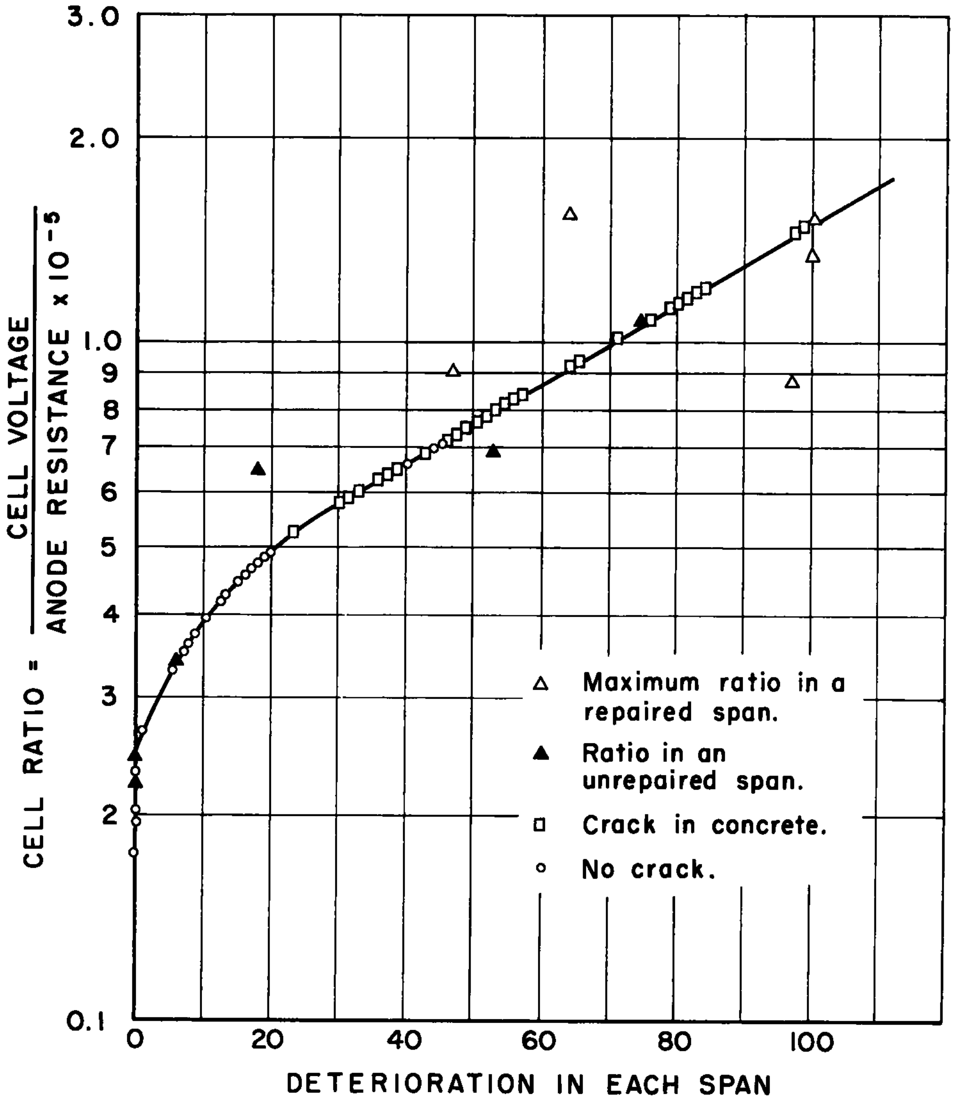

Figure 21. Concrete cracking against corrosion cell ratio.

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-

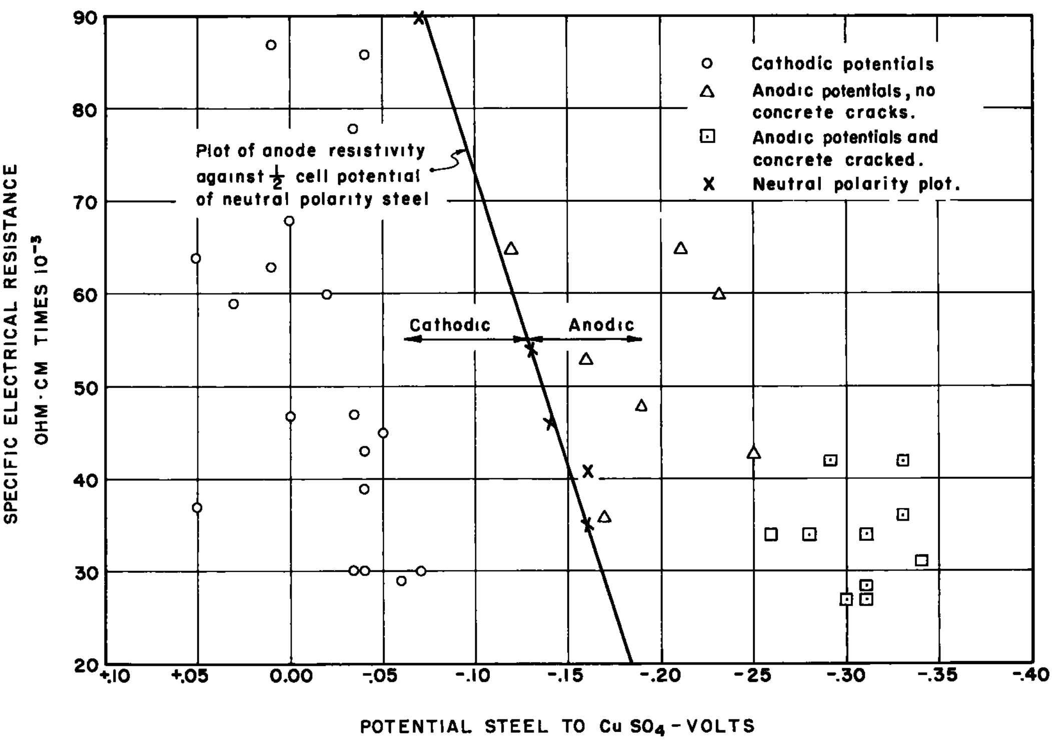

Figure 22. Half-cell potential of steel against resistivity of concrete.

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-

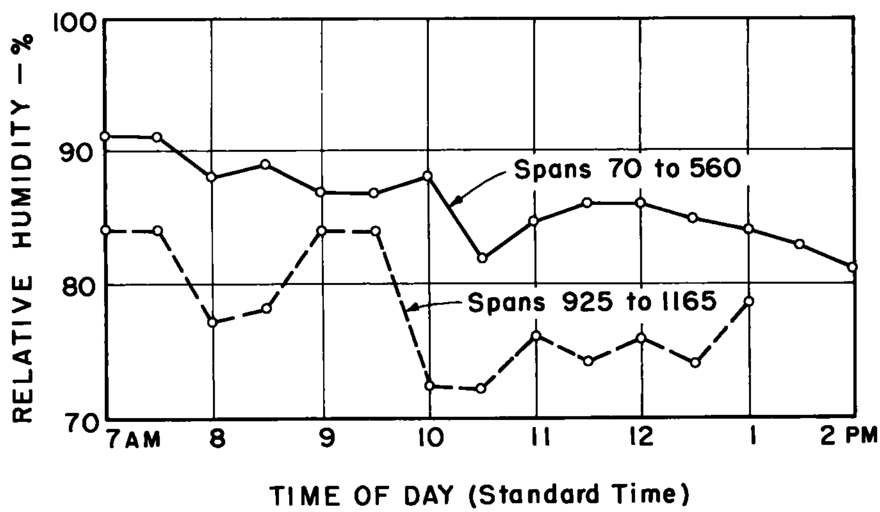

Figure 23. Relative humidity under bridge and shaded from sunlight, July-Oct. 1955.

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 rep-resentative of the deterioration of an average corrosion cell in the bridge. As the curv 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 th 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 reinforc-ing 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 vary-ing amounts of resistivity (specific electrical resistance of the concrete) as determine by the position of the crosses. Cathodic potentials fall to the left of the curve and anodi potentials to the right.

The chart shows that cracking in these spans occurred only when the anode resistiv-ity 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 meas-urements 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 deter-minations were not made concurrently, Weather Bureau records indicate that a differ-ential 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 unde 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, resistiv-ity 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-

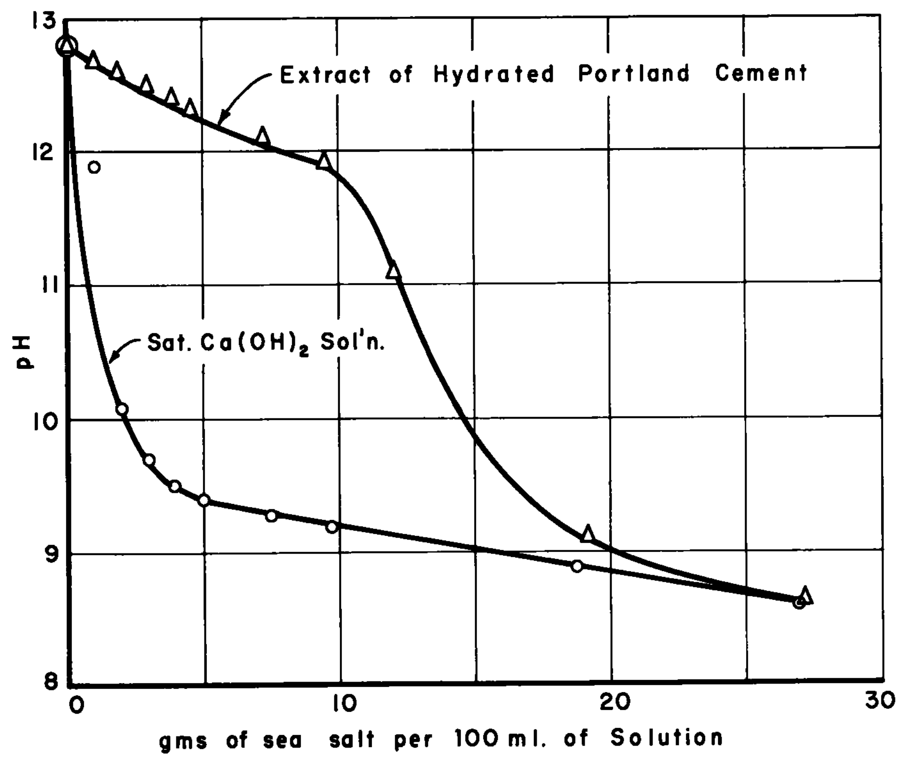

Figure 24. Effect of sea salt on pH of calcium hydroxide solution and hydrated Portland cement extract.

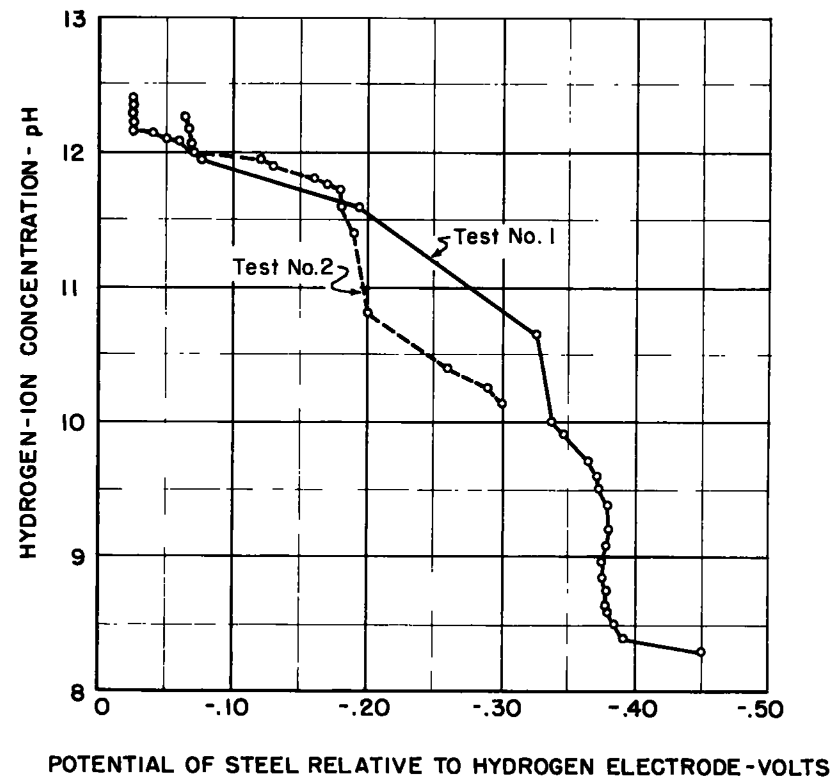

Figure 25. Potential of steel in cement extract solution of variable pH by addition of sea salts.

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 Mate 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 alkalies, 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 7½ 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 constan

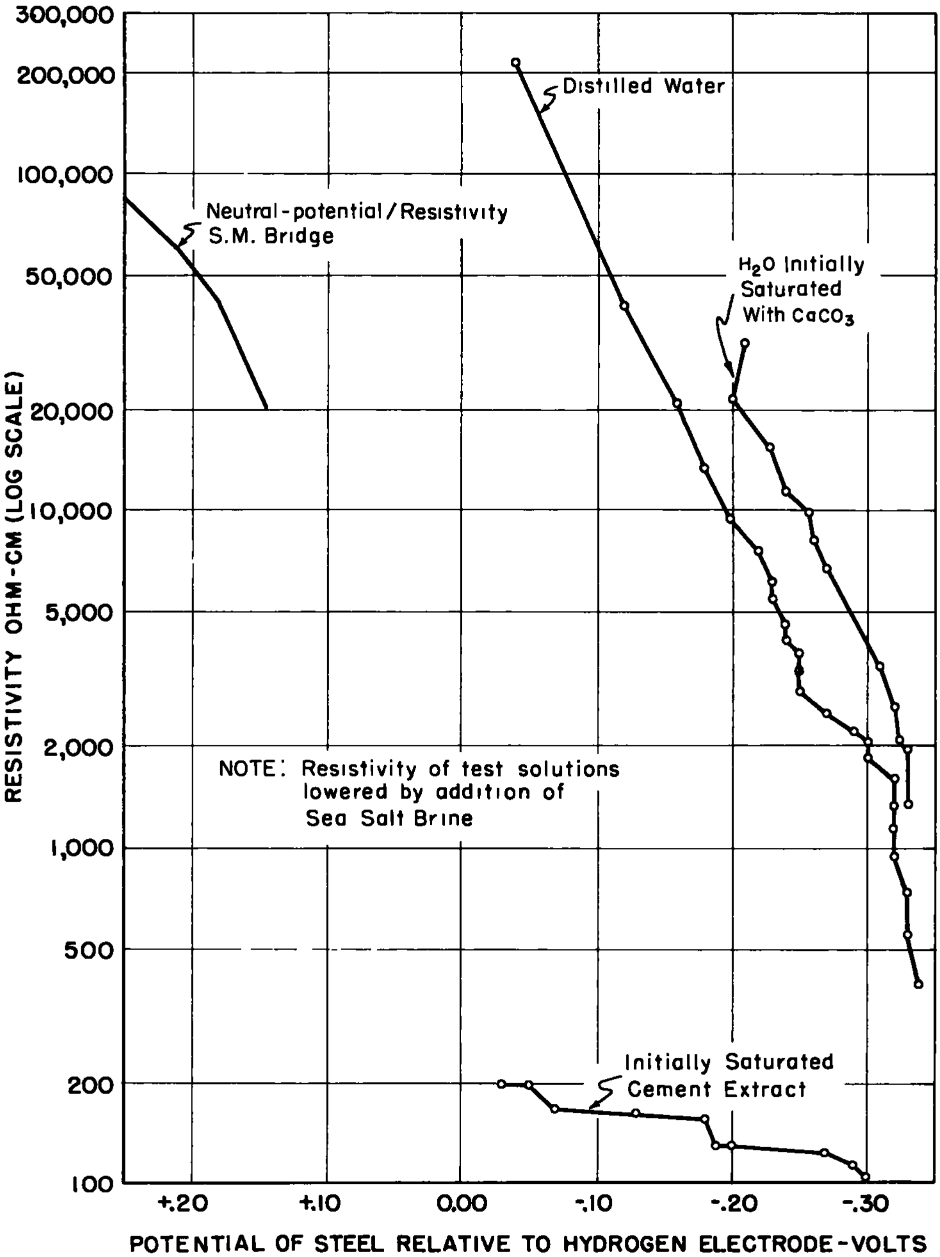

Figure 26. Potential of steel in solutions of variable resistivity by addition of sea salts.

Each sample was than 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 alkalies 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 potential 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

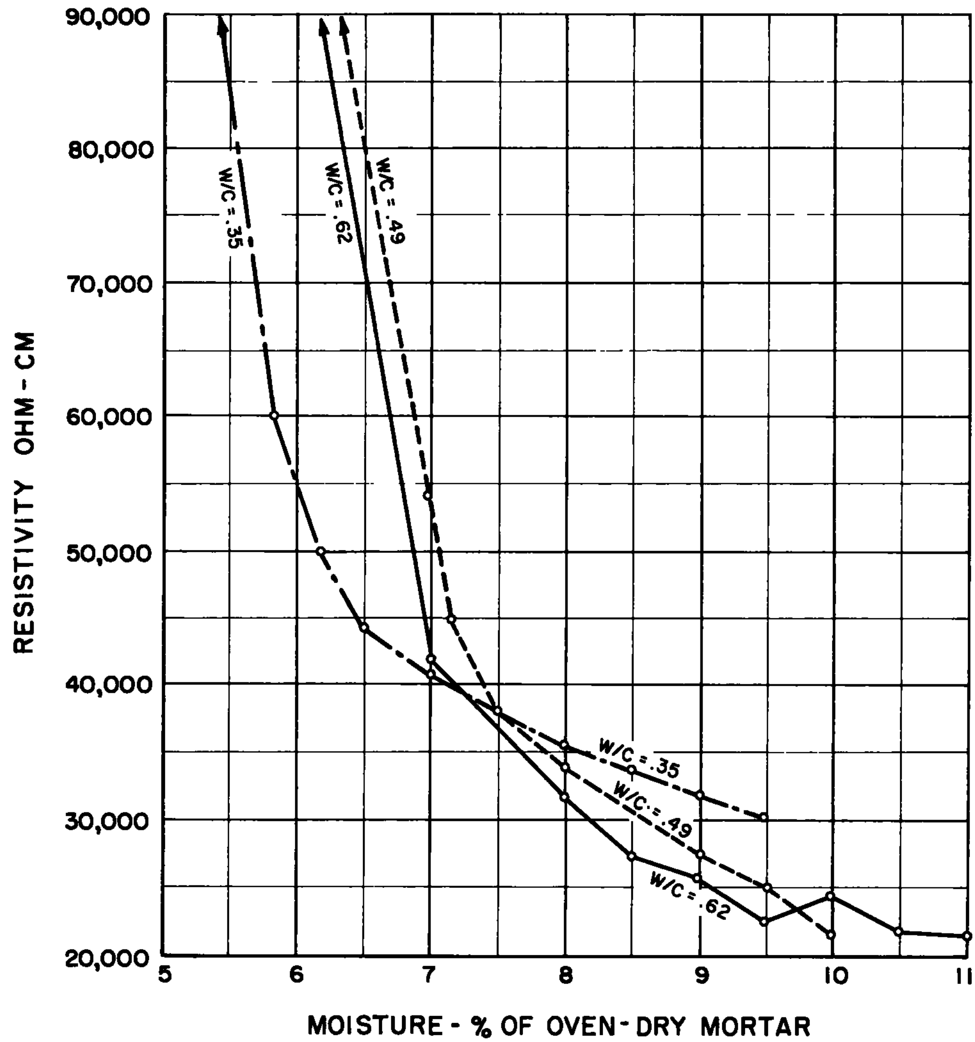

Figure 27. The effect of water cement ratio on the resistivity of mortar at various moisture contents (all salt contents averaged).

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 expressed 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

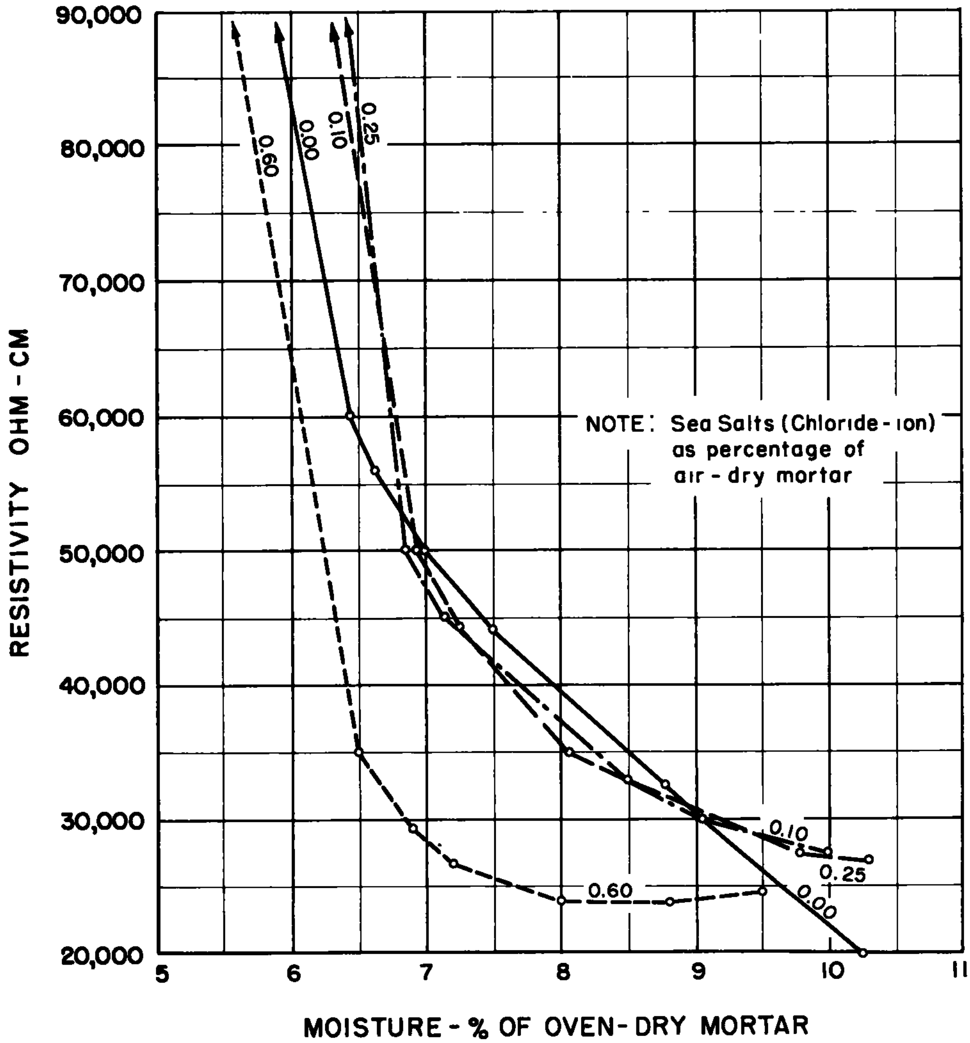

Figure 28. The effect of sea salts on the resistivity of mortar at various moisture contents (all water-cement ratios averaged).

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 3½ 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 4½-in. blocks of mortar containing two ¼- 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: Current Drawn, amp-days	10-Volt Series: Cell Reaction	2-Volt Series: Current Drawn, amp-days	2-Volt Series: Cell Reaction	0.5-Volt Series: Current Drawn, amp-days	0.5-Volt Series: 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

[a]Electrodes ¼-in. steel, 2 in. wide and imbedded 3 in. in concrete.

[b]Percent 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 Massachusetts, 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 oxyger 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. Accompanyin 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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