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

I. Method of Repair

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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

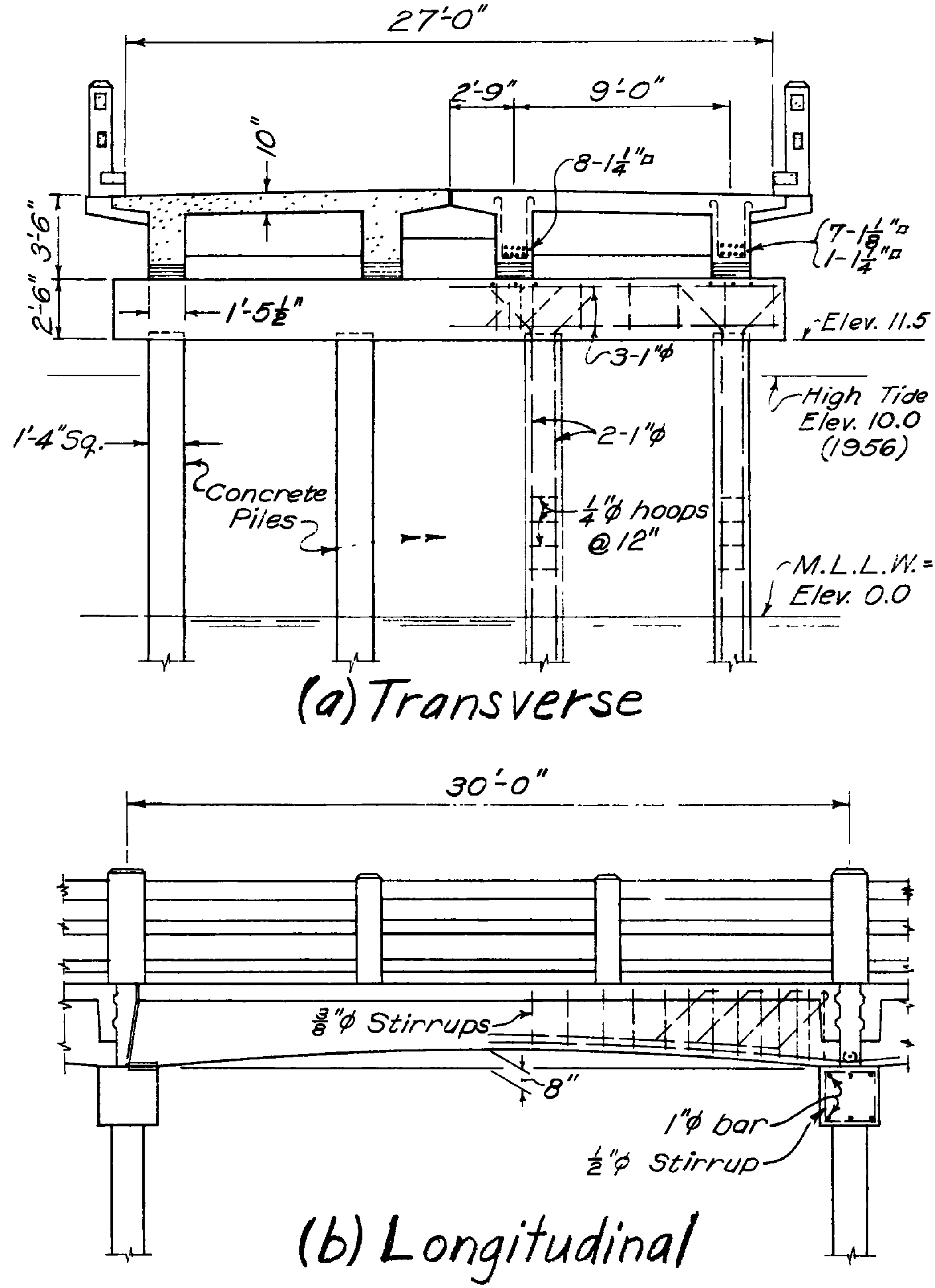

Figure 1. Typical sections of 30-ft span.

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.

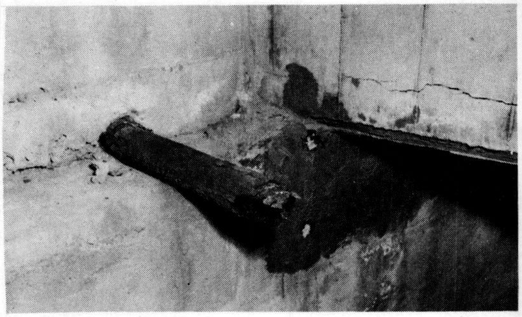

Figure 2. Typical condition of open cracks in beams. Projecting member is an iron pipe sleeve installed through end diaphragm during construction and now completely disintegrated.

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-

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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.

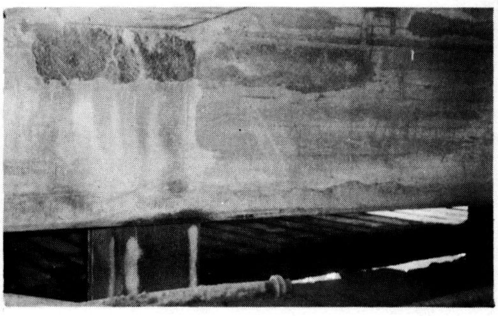

Figure 4. Typical crack in side of pile cap in plane of bottom reinforcement.

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.

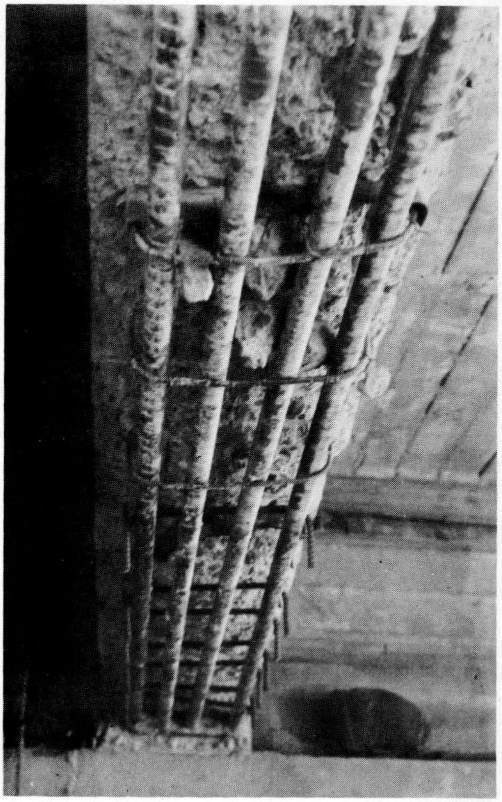

Figure 6. Beam bottom after chipping and before sandblasting, showing spotty and discontinuous areas of corrosion on reinforcement. Stirrup replacements at end of beam yet to be installed.

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

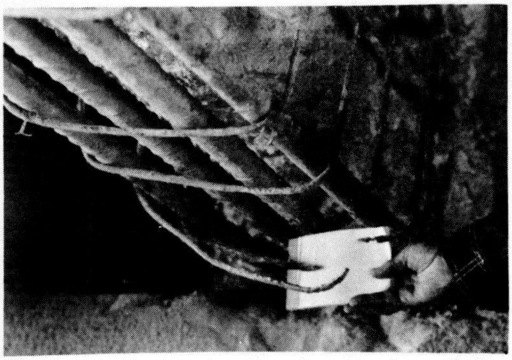

Figure 7. Tapered ends of severed 3/8-in. round stirrups show almost 100 percent loss of cross-section.

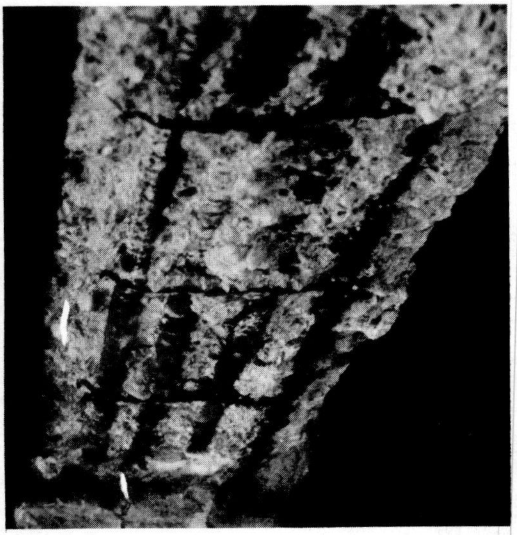

Figure 8. Area where deterioration has advanced to such degree as to spall off a considerable portion of the beam bottom.

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

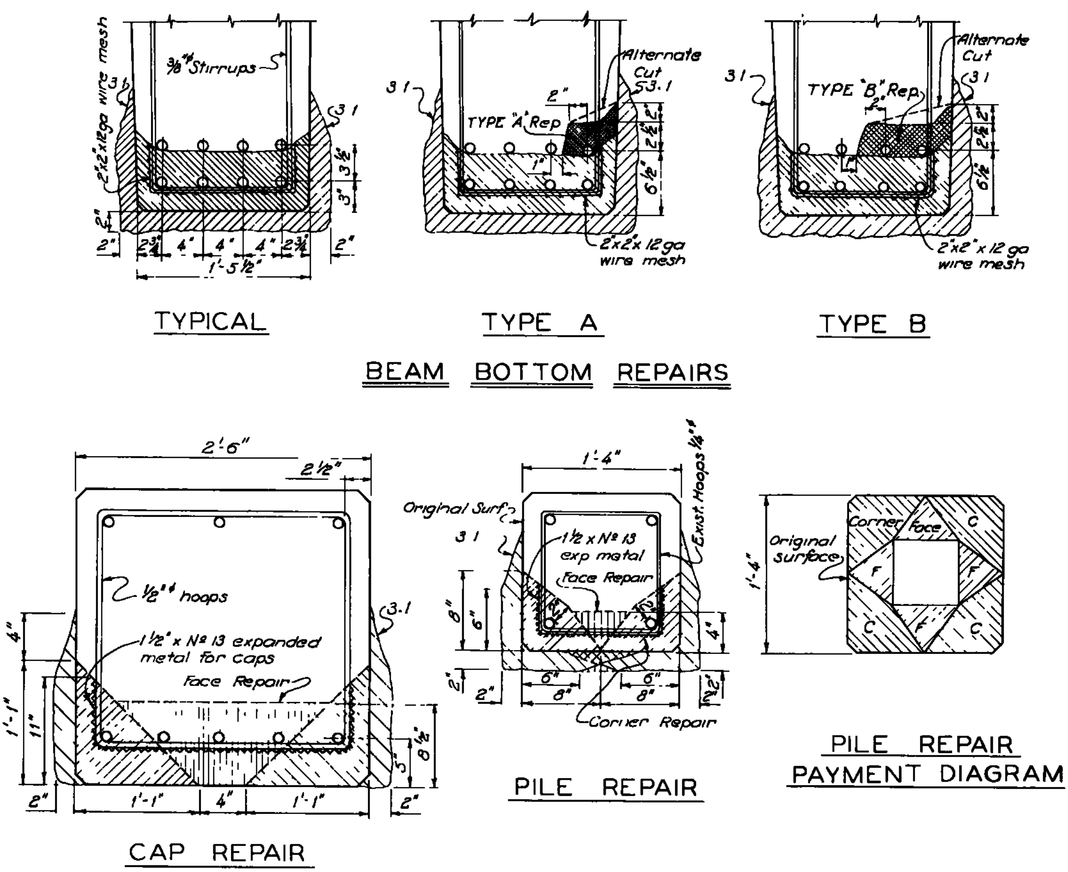

Figure 9. Typical cuts and details shown on contract plans.

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	REMARKS
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

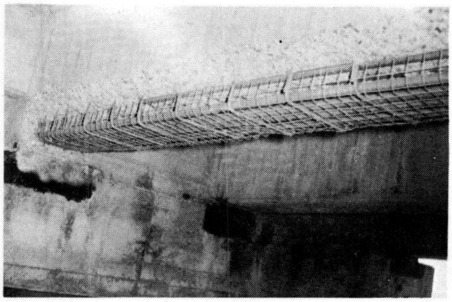

Figure 11. Beam bottom with sturrups replaced and wire mesh in place, ready for shotcrete.

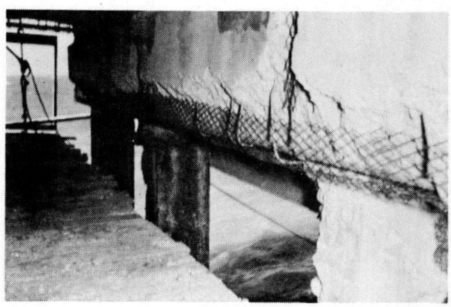

Figure 12. Cap corner ready for shotcrete. Note vertical chases cut to remedy corrosion on stirrup legs.

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 the 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 opposite 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-

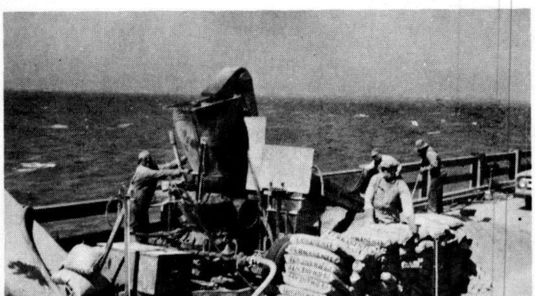

Figure 13. Contractor's equipment for batching, mixing, and shooting shotcrete.

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.

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.

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

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.