State of California
Department of Public Works
Division of Highways
Materials and Research Department

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IV-SM,Ala-105-B,A
Lab. W.O. 5006 R 54
(Part I)

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tate Highway Engineer
acramento, California

ear Sir:

Submitted for your consideration is:

An Investigation of

CAUSES OF CORROSION OF REINFORCING STEEL

in

SAN MATEO-HAYWARD BRIDGE

IV-SM,ALA-105-B,A

Study made by ..... Technical Section
Under general direction of ..... Bailey Tremper
Report prepared by ..... Bailey Tremper

Very truly yours,

F. N. Hveem

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Materials and Research Engineer

cc:EW
HCW
FWP
RHW
JWV

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TABLE OF CONTENTS

	Page
LIST OF FIGURES AND TABLES	
FOREWORD	
SYNOPSIS	
DESCRIPTION OF STRUCTURE	1
History	1
Maintenance	4
Nature of the Deterioration	4
Description of Repair Work	5
THEORY OF CORROSION	6
LABORATORY INVESTIGATIONS	7
Physical Tests	7
Compressive Strength	8
Absorption	8
Permeability	9
Transpiration	10
Composition of the Concrete	11
Cement Content	11
Grading of Aggregates	12
Water-Cement Ratio	12
Chemical Tests	13
Evidence of Sulfate Attack	13
Carbonation and Leaching	14
Chlorides	14
Effect of Sea-Salt on pH	16
Spacer Blocks	18
GENERAL OBSERVATIONS	21
DISCUSSION	21
UNANSWERED QUESTIONS	24
RECOMMENDATIONS	24
CONCLUSION	25
LIST OF REFERENCES	

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LIST OF FIGURES AND TABLES

Figure A	View of Bridge During Construction
Figure 1	Effect of pH of Aqueous Solutions on Rate of Corrosion
Figure 2	Air-permeability Apparatus, Transpiration Test
Figure 3	Effect of Sea-Salt on pH
Figure 4	Effect of Moisture on pH of Mortar Containing Sea-Salt
Figure 5	Distribution of Corrosion on Lower Bars of Beams
Table I	Compressive Strength
Table II	Absorption
Table III	Permeability to Air
Table IV	Transpiration
Table V	Cement Content
Table VI	Grading of Aggregate
Table VII	Magnesia and Sulfuric Anhydride in the Concrete
Table VIII	Leaching and Carbonation
Table IX	Chlorides in Core Samples
Table X	Chlorides in Concrete Adjacent to Steel
Table XI	Effect of Sea-Salt on pH
Table XII	Composition of Spacer Blocks
Table XIII	Physical Tests of Spacer Blocks

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FOREWORD

A study of the causes leading to the corrosion of reinforcing steel in the San Mateo-Hayward Bridge has been undertaken in two phases by the Materials and Research Department. This report presents data on one phase, that of physical and chemical properties of the concrete and has been conducted by the Technical Section. The second phase deals with actual measurements of the magnitude and direction of the electrical, potential and current and is being conducted by the Structural Materials Section. The findings of that section will be presented in a second report.

By agreement between the Bridge Department and the Materials and Research Department, the present investigations were not planned to be exhaustive. The purpose, rather, was to conduct pilot studies that would be of assistance in determining whether or not more complete investigations would be warranted.

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SYNOPSIS

The concrete of the San Mateo-Hayward Bridge, a structure 7 miles in length across San Francisco Bay, constructed in 1928-9, is severely cracked due to corrosion of reinforcing steel.

This report deals with the physical and chemical properties of concrete in the beams, caps and piles. In the bents sampled it appears that the concrete was mixed with a water-cement ratio that was too high to assure durability under the conditions of exposure. The concrete is relatively permeable and has absorbed salts from the sea water. Sulfates have attacked the concrete to the extent that the present strength is no higher than was reported at the age of 28 days.

Salts that have concentrated in the concrete have lowered the pH and have increased electrical conductivity. As a result the concrete, in its present condition at least, affords less than normal protection against corrosion of the reinforcement.

Precast mortar blocks, installed to support reinforcement above the bottom of beam forms, have very little bond with the adjacent concrete. Medium to severe corrosion has occurred more frequently in the vicinity of the blocks than elsewhere along the bars.

Many pertinent questions remain unanswered by the results of this investigation.

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DESCRIPTION OF STRUCTURE

The San Mateo-Hayward Bridge is a two-lane concrete pile trestle, 7 miles in length, crossing the southern arm of San Francisco Bay about 15 miles south of San Francisco. Except for a lift span and four truss spans totalling 1503 feet in length, the bridge consists of concrete beam spans of which 116 are 35 feet in length and 1054 are 30 feet in length. The roadway is 27 feet wide between curbs.

The typical bent consists of four precast reinforced concrete piles supporting a cast-in-place reinforced concrete cap. The caps support precast reinforced concrete deck sections each of which is one-half roadway width. Precast curb and rail sections complete the trestle.

Piles vary from 16" x 16" to 24" x 24" in cross-section and contain four reinforcing bars of varying sizes. One quarter inch (1/4") hoops are spaced at 3-inch centers in the upper and lower 4 feet and at 12-inch centers throughout the remainder of the pile. Caps are 30" x 30" in section and contain eight 1" round bars, three each near the top and bottom and two bent bars tied with 1/2-inch hoops. The deck section consists of a roadway slab 10 inches thick tapering to 6 inches at the outer edge and 8 inches at the roadway center line. Each unit was cast monolithically with two beams. The interior beam is 18 inches wide, 44 inches deep at mid-span. The exterior beam is 2 inches less in depth at all points. Each beam contains two layers of four reinforcing bars of ovoid shape equivalent in area to 1-1/8" and 1-1/4" square bars. There are nominally 31 3/8" stirrups per beam.

HISTORY

The bridge was constructed under private ownership in 1928-9 and was opened to traffic in March, 1929. It was operated as a privately-owned toll crossing until September 12, 1951 when it was acquired by the State of California.

Construction records available to the State are incomplete but the following information has been obtained.

Precast members were constructed in Alameda and were transported to the bridge site on barges. The nominal mix used in the piling was 1:1-1/2:3 but the actual reported mix was approximately 1:2:3, presumably by volume. Celite at the rate of 2 pounds per sack of cement was added after the first month. Piles were cast between March 1 and September 21, 1928, however 197 additional piles were manufactured subsequently as replacements because of breakage in handling, transportation and driving. Time of mix was 1-1/2 minutes up to August 14 and 2

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minutes thereafter. A chart is on file showing slump tests and 28-day compressive strengths of 170 test cylinders. Slumps varied between 3-1/2" and 6-1/2" with the majority in the 4-1/2" and 5-1/2" range. Substantially all compressive strengths are shown to be in the range of 4000 - 5000 psi. Total load on the cylinders was recorded to the nearest 1000 pounds, indicating possibility that a gage-equipped hydraulic jack was used in testing. The record shows that the cement factor was from 7.2 to 7.6 sacks per cubic yard until May 1, after which it was from 6.6 to 6.8 sacks per cubic yard. A summary for the piles shows the following quantities:

Concrete	28,790 cu. yd.
Cement	49,264 barrels
Sand	14,866 cu. yd.
Gravel	20,866 cu. yd.
Celite	3,419 sacks
Average Cement Factor	6.9 sacks per cu. yd.

Pile driving was completed on October 25, 1928. Piles were cut off to grade but in some cases it was necessary to cast extensions to the driven piles.

Precast deck units were constructed between April 3 and October 24, 1928. A photo of the casting yard shows that burlap was used in curing precast members. The resident engineer's report states that deck units were moved by barge as soon as "cast and poured" as there was no provision for storage at the yard. The cement factor is shown as between 6.0 and 6.2 sacks per cubic yard during the first 9 days, after which it was reduced to 5.2 to 5.4 sacks per cubic yard. The record shows results of daily tests for slump which in the main were between 3 inches and 5 inches. A record of test cylinders was found which gave results of 7-day tests of one cylinder per day for the first 13 days. For the first ten days the compressive strength at 7 days varied between 2050 and 2550 psi, during which time the nominal 6-sack mix was used. On the succeeding three days, with a nominal 5.3-sack mix, the reported 7 day strengths were 1595, 1698 and 1796 psi. No other record of strength tests of concrete in the deck units has been found. A summary shows:

Concrete	50,755 cu. yd.
Cement	68,424 barrels
Average Cement Factor	5.36 sacks per cu. yd.

No record has been located containing data for the cast-in-place caps.

Portland cement was Golden Gate brand, manufactured by Pacific Portland Cement Company, Redwood City. Inspection

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reports by Robert W. Hunt and Co. show compliance with ASTM specifications C 9-21. Magnesia varied from 1.73 to 2.01 percent and averaged 1.83 percent. Sulfuric anhydride varied from 1.51 to 1.71 percent and averaged 1.61 percent. The following chemical analysis has been furnished by the manufacturer as typical of the cement furnished on the work:

SiO_2	21.31
Fe_2O_3	2.54
Al_2O_3	6.29
CaO	65.01
MgO	1.71
SO_3	1.53
Loss on Ignition	0.68
C_3S	52
C_2S	22
C_3A	12
C_4AF	8

Concrete aggregates were sand and gravel produced by Grayhouse and Montagne near Pleasanton. Grading limits as follows were adopted for the coarse aggregate:

Percent	Passing	1-1/4"	square	opening	100
"	"	3/4"	"	"	55 - 75
"	"	1/2"	"	"	40 - 60
"	"	No. 4	"	"	0 - 15
"	"	No. 8	"	"	0 - 5

The resident engineer reported that a high percentage of the gravel would pass 1 inch.

Records that have been found concerning the grading of the sand are very incomplete. The following was reported to be typical of production at the plant:

Percent	passing	No. 4	100
"	"	No. 20	50
"	"	No. 50	15
"	"	No. 100	4

A curve plotted from the above data shows approximately the following grading on present-day sieves:

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Percent passing	No. 4	100
" "	No. 8	85
" "	No. 16	65
" "	No. 30	35
" "	No. 50	15
" "	No. 100	4
Fineness modulus		2.86

MAINTENANCE:

It is reported that longitudinal cracks were noted in pil caps and beams shortly after construction. The owners commenced repairs with shotcrete in 1936. A heavy bituminous coating was applied to about 70 percent of the beam stems, caps and exposed portions of the piles beginning in 1936 and continuing into 1940 The coating evidently was ineffective in preventing further deterioration and in 1949 more extensive repairs with shotcrete were commenced. A bituminous mix was placed on the deck east of the steel spans to correct inequalities in surface.

After acquisition by the State the program of restoration has continued, partly by State forces and partly by contract. A pilot contract involving 48 spans was completed in 1953 at a con track price of $123,061. Recently a contract has been awarded for the repair of 200 spans at a bid price of $484,160. The [illegible] of repairs is indicated on approximately 700 of the 1160 spans. The estimated total cost is between $1,500,000 and $2,000,000.

NATURE OF THE DETERIORATION

The water of San Francisco Bay at the bridge site contai approximately 17,000 ppm of chloride-ion or about 86 percent of the concentration of average ocean water.

Recent surface inspections of a few spans indicate little if any, visible deterioration of the concrete as a result of sul fate attack. The pulse velocity in uncracked concrete, measured with a soniscope was found to vary between 12,250 and 14,600 fe per second in beams and between 14,000 and 14,800 feet per se in piles. These values are considered to be indicative of conc in good to excellent condition.

Cracks have formed parallel to the direction of the main reinforcement. The cracks open progressively and ultimately result in spalling of fragments from the surface. In substantiall all cases cracking and eventual spalling is found to be connecte with the development of rust on the reinforcement.

Cracks develop in the piles typically in a vertical direction about 3 inches from the corners and are present in the upper 8 feet, or above mean tide. Occasionally cracks extend downward to about low tide. Cracks develop in the caps in a

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horizontal direction opposite the reinforcement and generally in proximity to the lower face. Cracks develop in the beams opposite the lower line of reinforcement.

Fine cracks are not readily visible because of the presence of the bituminous coating. However, as corrosion advances bulging due to rust pressure becomes evident and, as noted above, eventually results in the detachment of fragments about 2 inches in thickness and of exposure of the reinforcement.

DESCRIPTION OF REPAIR WORK

Under State direction, all areas of piles, caps and beams in which corrosion is evident or is suspected are chipped with pneumatic tools to expose the reinforcement on all sides, and to a sufficient depth beyond the inner surface to permit thorough cleaning and subsequent filling with shotcrete. Chipping is continued along the length of the bar until it is judged that corroded areas have been fully exposed. Frequently the entire length of beams is chipped.

Upon completion of chipping, the area is sand blasted until the steel is bright and loose fragments of concrete are detached. Wire mesh or expanded metal reinforcement is attached to existing reinforcement where considered necessary. Shotcrete is then placed and extended about 1 inch beyond the original surface. The shotcrete is then sprayed with pigmented concrete curing compound.

Shotcrete is composed of a 1:3 mix of Type III cement of low C_3A content and a rather coarsely graded sand. The use of a 1:4 mix is under investigation. The 1:3 shotcrete has a compressive strength of 7000 to 8000 psi at 28 days.

The majority of corrosion in the beams is found in the lower line of bars and in the lower 3 to 6 inches of the stirrups. Rust is more prevalent on the lower half of the bar surface. Accumulations 1/8 to 1/4 inch in thickness are found in severely affected areas. The area of the main bars is not found to have been reduced sufficiently to warrant replacement but a few stirrups are seriously weakened. The lower portion of stirrups is replaced by welding a new section when judged to be necessary.

A considerable portion of earlier repair work under private ownership has been found to have been ineffective in arresting further corrosion. It is now believed that insufficient care had been taken to remove corrosion products thoroughly from the steel before placing shotcrete. Thorough sand blasting over the entire surface of exposed steel is now stressed strongly.

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THEORY OF CORROSION

It is generally believed that corrosion of metals is electrochemical in nature. This means that corrosion proceeds through the operation of cells consisting of two dissimilar metals in contact in an aqueous medium which forms the electrolyte. While we ordinarilly think of such a cell as being up of two distinct metals, copper and zinc for example, local cells may be set up by minor variations in chemical composition within the same metal or by a discontinuity in an oxide film as mill scale on steel.

As applied to iron or steel, corrosion involves the soluti of iron as ferrous-ions at the anode of a cell and the formation of hydrogen-ions and hydroxyl-ions at the cathode. The relative number of hydrogen-ions and hydroxyl-ions that are formed depend on the degree of acidity or alkalinity, that is, the pH value, of the aqueous medium (electrolyte) of the cell. Hydrogen-ions may unite to form hydrogen gas or they may unite with oxygen, if it is present, to form water. Hydroxyl-ions may migrate unti they meet ferrous-ions from the anode and form ferrous hydroxide Ferrous hydroxide may be reacted upon by oxygen or carbon dioxi in air to form complex, insoluble compounds (rust).

The electrochemical process causing corrosion is retarded, or it may be stopped completely, by polarization of the cathode by hydrogen or by the formation of a non-conducting film of corrosion products at the anode. Oxygen, if available, tends to depolarize the cathode by uniting with the hydrogen. Under certain circumstances the rate at which corrosion proceeds is dependent mainly on the quantity of oxygen that reaches the cathode. The character of the film of corrosion products (rust) its conductivity and its place of formation relative to the area depend on conditions that are too involved for discussion here.

The rate at which corrosion proceeds is influenced greatly by the pH of the medium surrounding the metal. Figure 1 shows curves illustrating this effect. The upper chart is reproduced from Reference (1) and shows that rate of pitting of steel in water containing a controlled amount of oxygen. The lower chart is from Reference (2) and expresses the rate of corrosion in general terms. Both charts illustrate the same principle. In waters more acid than pH 4 or 5 hydrogen is evolved freely and corrosion will proceed whether oxygen is present or not. In the region between pH 4 or 5 and about 10 the rate is substantially constant for a given rate of supply of oxygen at the surface of metal where it can act as a depolarizer. Above a pH of about 10 the rate of corrosion diminishes with increasing pH

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and at pH 13 is substantially inhibited.

In the body of normal concrete, the pH of the water in the pores is of the order of 12.5 to 13. Concrete thus forms a medium which distinctly inhibits corrosion. Furthermore the texture of concrete retards the penetration of atmospheric oxygen. When reinforcement does corrode there must be some abnormality present in the concrete. If an impressed current of substantial voltage is present, as from leakage from DC lines, electrolysis may take place and corrosion may proceed regardless of the normal protective value of the concrete. A discussion of electrolysis is reserved for Part II of this report. Part I is concerned with the presentation of data showing the processes by which concrete of the San Mateo-Hayward Bridge has lost its normal protective value.

LABORATORY INVESTIGATIONS

Early in 1953 two 2 x 2 x 4-inch prisms were sawed from a fragment of concrete removed from a beam during repair work. The compressive strength of the prisms was found to be 3660 and 3865 psi respectively. The strengths were believed to be low considering the age of the concrete and to indicate rather poor quality considering the requirements for the conditions of exposure. Additional physical and chemical tests have since been made and are described below.

PHYSICAL TESTS

Twenty 2-inch cores, 8 inches in length were obtained by drilling horizontally into beams, caps and piles. Twelve cores were taken from four beams, four cores from two caps and six cores from three piles all in a single span of the bridge. The cores were taken from positions selected to avoid the main reinforcing steel. In beams the position was about 10 inches above the lower face. Cores were placed in polyethylene bags and then in metal cans with taped covers for shipment to the laboratory. While core drilling was in progress, State operators prepared two shotcrete panels 5 inches thick, one of which was placed against a vertical surface and the other against a horizontal surface. Cores were taken from these panels at a later date. Large sections of old shotcrete in place on piles were shipped to the laboratory and cores were taken from them. There were on hand in the laboratory sections of 2-inch vertical cores from a cast-in-place pile of a recent contract. These specimens were used in some tests for purposes of comparison. The cores were sawed, with a diamond saw running in water, into lengths

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suitable for the tests described below.

Compressive Strength:

Portions of the core samples were sawed into 4-inch lengths, were soaked in water for 7 days and then were tested for compressive strength. Results are given in Table I.

TABLE I

Compressive Strength of 2 x 4-in. cores, PSI

Beams	Caps	Piles	Old Shotcrete	New Shotcrete Vertical	New Shotcrete Horizontal
3380	2920	4760	6740	9920	10390
2900	2280	4440	10190	9950	10930
3320		4040			
2630					
3040	2600	4410	8470	9740	10660

Absorption:

Prior to making compressive tests, the specimens were dri to constant weight at 220°F and then immersed in water for 5 hou Values of absorption after various periods of immersion are in Table II.

TABLE II

Absorption of 2 x 4-inch Cores in Cold and Boiling Water
Values are in Percentage by Weight of Oven Dry Specimens

	Beams	Caps	Piles	Old Shotcrete	New Shotcrete V*	New Shotcrete H*
Cold Water, 24 hours	6.73	6.79	6.47	2.53		
	6.29	5.78	6.15	2.51		
	6.47		5.55			
	6.00					
	6.11					
Ave.	6.32	6.29	6.06	2.52		
Cold Water, 48 hrs. Ave.	6.40	6.38	6.12	2.60		
Cold Water, 7 days, Ave.	6.55	6.55	6.25	2.76		
Cold Water, 7 days plus boiling water, 5 hrs. Ave.	6.08	7.48	6.74	2.81	6.53	5.86

*V, Placed vertically; H, placed horizontally

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Permeability to Air:

Sections of cores 1 inch in length were centered in 3-inch diameter pipe sections 1-1/4 inches long with one end of the core in a plane with one end of the pipe. The annular space was filled with a rosin-paraffine mixture. The assembled specimens were clamped between gaskets and steel plates in such a manner that air under pressure in one face could pass through the core and be discharged freely into the atmosphere at the other face. Figure 2 shows the apparatus used in these tests.

Permeability was determined on some specimens in a room-dry condition and also after storage in the moist room for several days. They were somewhat less permeable in a moist condition but since all were not tested after moist storage values are reported on the room-dry condition only. A few specimens were found to have small fractures extending full depth and the flow of air through them was too rapid for measurement. Coefficients of permeability were not computed. Results are reported as drop in pressure in 30 minutes from the original pressure of 30 psi. The latter values seem to be adequate to indicate the range in permeability. Results are given in Table III.

TABLE III

Permeability to Air of 1-inch Sections Sawed from 2-inch Cores. Values are drop in pressure from 30 psi in 30 minutes

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

Permeability to Water:

At the conclusion of the air permeability tests the confining rings were sealed at the bottom to cylindrical glass vessels. The space above the core section was filled with water thus subjecting the specimen to a head of about 1/4 inch. The specimens were stored in the moist room for 6 weeks after which they were examined for passage of water through the concrete. In most cases no water passed through but a trace passed through one pile specimen and 0.2 ml. and 4 ml. respectively passed through two specimens from caps.

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Transpiration:

The rate of flow of water vapor through capillaries in the concrete was investigated by means of the test shown in Figure 2. In this test sections of cores 3 inches in length with the lower 1 inch immersed in water were subjected to the evaporative effect of moving air. The assembled specimens were filled with water through a small opening in the metal disk whi- was then sealed. The entire assembly was then weighed to the nearest 0.1 gram. Twelve units were set up at the same time on a table in the path of an air current from a fan. The specimens were weighed daily to determine loss by transpiration. The position of the specimens in the table was changed daily to equalize any difference in evaporating conditions that might be present.

The cores were room dry before the start of the test. Evaporation losses were relatively low during the first day but reached a peak after two days and then decreased gradually duri- the next few days. After 6 or 7 days the rate of loss became relatively constant. Representative results are shown in Table IV.

TABLE IV

Rate of Transpiration in Grams per Day for the 24 Hour Period Preceeding the Test Age Shown

Test Age, Days	Beams	Caps	Piles	Old Shotcrete	New Shotcrete V*	New Shotcrete 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		0.8		0.1	0.3
	3.6					
	3.2					
	5.5					
	3.5	4.0	1.1	0.0	0.2	1.0
8	2.5	2.8	1.5	0.1	0.2	1.5
	1.0		0.6		0.2	0.2
	2.7					
	2.1					
	3.8					
	2.4	2.8	1.1	0.1	0.2	0.9

*V, placed vertically; H, placed horizontally

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Summary of Physical Tests of Cores:

Test results reported in Tables I to IV inclusive illustrate the poor quality of concrete in beams and caps as shown by low compressive strength, high absorption and high permeability to air and water vapor. The results uniformaly show the quality of the concrete in the caps to be somewhat lower than that in the beams. The tests indicate higher quality in the piles. In comparison with the original concrete, the older shotcrete gave greatly superior results. Cores from newly made shotcrete specimens were slightly inferior to those from older material doubtless due to less complete hydration of the cement. Shotcrete placed against a vertical surface was superior to that placed against a horizontal surface.

COMPOSITION OF CONCRETE

No construction record has been found to show the cement content of the concrete used in the caps. It was desirable to check the construction records of piles and deck sections by determinations in the laboratory.

Cement Factor:

Large fragments from the various members were broken down in the laboratory to separate coarse aggregate from mortar. Calcium oxide and loss in ignition were determined on pulverized mortar samples. These results were used to compute the ratios of cement to aggregate. Using assumed water-cement ratios, the cement factor in sacks per cubic yard were computed. The results are shown in Table V, together with values given in the construction records.

TABLE V

Member	Cement Content, Sacks per Cubic Yard Construction Records	By Analysis
Piles	6.9 - 7.4	7.1
Beams	5.3 - 6.1	5.3
Caps	- - -	5.4

The results of analysis are in good agreement with the reported cement factors for piles and beams. It may be assumed therefore that the analytical results for the caps is substantially correct.

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Grading of Aggregate:

Portions of the mortar as prepared for chemical analysis but before pulverization were heated with dilute hydrochloric aci repeatedly until all of the cement appeared to have been dissol Sieve analyses were made of the coarse and fine aggregates. The results are shown in Table VI. The sieve analysis of the sand as originally determined showed a rather high percentage passing No. 200. It was felt that this result was due to the presence of Celite in the concrete, to attrition of the particles during hand ling and possibly to the presence of undissolved cement particles The grading shown in Table VI has been adjusted arbitrarily to 2 percent passing No. 200. Some particles of coarse sandstone were reduced to sand sizes but no attempt has been made to correct for this effect.

TABLE VI

Grading of Aggregates

Coarse Aggregate	By Analysis	As Specified
% Passing 1-1/4"		100
1"	100	
3/4"	85	55-75
1/2"		40-60
3/8"	38	
No. 4	0	0-15
No. 8		0-5

Fine Aggregate	By Analysis	Reported at Plant
% Passing No. 4	100	100
No. 8	75	85
No. 16	48	65
No. 30	28	35
No. 50	10	15
No. 100	4	4
No. 200	2	
Fineness Modulus	3.35	2.86

Water-Cement Ratio:

Based on a maximum size of 1 inch and a probable slump of 4 to 5 inches it can be estimated with reasonable assurance that the concrete as mixed contained about 40 gallons of water per cubic yard. (Reference 3). Using the cement factors as determined in this investigation the water-cement ratios of the concr are computed to be as follows:

Beams and Caps	7.5 gals. per sack
Piles	5.7 gals. per sack

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CHEMICAL TESTS

In addition to tests for cement content, analytical determinations were made for other constituents. Samples used consisted of mortar prepared from core samples in some cases and from large detached fragments from the structure in others. The results are reported below.

Evidence of Sulfate Attack:

Lea and Desch (4) state:

> "The chemical action of sea water on concrete is mainly due to the presence of magnesium sulphate. . . . Magnesium reacts with free calcium hydroxide in set Portland cement to form calcium sulphate, at the same time precipitating magnesium hydroxide; it also reacts with hydrated calcium aluminate to form calcium sulpho-aluminate. These are the actions primarily responsible for the chemical attack of concretes by sea water. . . . Analyses of concretes decomposed by sea water invariably show the presence of a high content of sulphates. The latter tend initially to accumulate in the concrete, but under the long continued action of sea-water their content may decrease again as clacium sulphate is slowly leached out."

Thus a high content of magnesia and, to a less extent, of sulfate, is an indication that the concrete has been attacked by sulfates in the sea water. Analytical determinations are given in Table VII.

TABLE VII

MgO and SO_3 in concrete. Values expressed as percentages by weight of the contained cement

	MgO	SO_3
Beam	8.5	3.23
Gap	8.9	2.17
Pile	7.0	2.45
Original Cement*	1.87	1.55

*As given in inspection reports of cement shipped to the work.

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It is evident that the symptons of sulfate attack are present in all three members of the bridge.

Carbonation and Leaching:

The major sources of alkalinity in the water contained in concrete is due to alkalies contained in the cement and calcium hydroxide formed during hydration of the cement. The former are highly soluble and may be leached from the concrete. The latter is formed in relatively large amounts and is only slightly soluble. Under very severe leaching conditions however, a large part of the calcium hydroxide may be removed. Leaching may therefore remove the alkaline constituents of concrete and in this way lower the pH. When carbon dioxide from the atmosphere comes in contact with calcium hydroxide in solution calcium carbonate is precipitated and in connection with leaching, the pH of the contained water can be lowered to values approaching 7 (5). Although serious reduction in pH by these means is unlikely in members of the size used in the structure the possibility of such action was nevertheless investigated. A core from a beam was cut into sections 1 in. long and the mortar was separated. 60-gram portions were placed in flasks with 200 ml. of freshly boiled distilled water. The flasks were stoppered and agitated occasionally for 7 days. It was found that there was ample calcium hydroxide available in all sections of the core to maintain the pH of these rather dilute solutions well above 12, although there were indications that carbonation had progressed a short distance into the outermost section. Detailed test data are given in Table VIII at the end of this report.

Chlorides:

Chlorides are the principal salts in sea water. The chloride radical constitutes more than one-half of the contained salt. The analytical determination of chlorine-ion is relatively simple and a large number of analyses of concrete for this constituent were made during this investigation. For convenience the expression "chlorides" will be used to denote the chlorine-ion or chloride radical. Reported values of chlorides if multiplied by the factor 1.8 will represent the approximate equivalent [illegible] of sea-salt. Chlorides were found in sawed sections of cores as shown in Table IX.

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TABLE IX

Chlorides in Sections Sawed from Cores

Letters refer to position of 1-inch sections in relation to the surface of the member, (a) representing the outer section. Values are percentages by weight of the room-dry mortar

Member	Position (a)	(b)	(c)	(d)	(e)	(f)
Beam	0.11	0.06	0.05	0.02	0.01	0.01
Caps	0.09	0.07	0.08	----	0.06	0.05
Pile	0.29	0.21	0.12	0.15	0.06	----
Pile	0.31	0.23	0.15	0.08	0.04	0.04

It will be noted that the concentration of salt is heavier in the piles than in the cap or beam as would be expected. The concentration diminshes with distance from the surface and is reduced to about one-half at a distance of 2 to 3 inches, the position of the main reinforcement. The cores, however, were not taken in close proximity to the reinforcement. To determine the chloride content in the region of the steel a number of fragments were detached from the structure and analysed. The fragments were from concrete in contact with steel that was corroded and also, except for piles, from concrete in contact with substantially non-corroded steel. The results are given in Table X.

TABLE X

Chlorides in Concrete Adjacent to Reinforcing Steel
Values in Percent by Weight of Room-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	

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The results indicate to a limited extent that corrosion has occurred when the concentration of salt has exceeded an undefined minimum. It is also true, however, that the concrete was cracked in the region of severe corrosion and the high content of chlorides may simply be the result of conditions that are more favorable to the accumulation of salt. At any rate, the data show that the concrete has absorbed substantial quantities of salt and that the concentration varies within rather wide limits. It seems likely that the salt entered both from wave splash or spray and by capillary rise and was concentrated near the surface by evaporation. The application of the bituminous coating undoubtedly greatly retarded the further accumulation of salt although it is unlikely that it prevented it completely. It does not seem probable that the coating had an effect on the rise of salt water through capillary action except as the rate was reduced by a lowered rate of evaporation from the surface. It should be noted that the construction of the bridge was such as to result in an unbroken path of concrete through which capillary rise would be uninhibited.

Three samples of heavy rust deposits on the reinforcement were found to contain 0.18, 0.28 and 0.32 per cent of chlorides respectively. These values are percentages by weight and on a volume basis, relative to the mortar of the concrete, would be several times as great.

A fragment from the hand rail at one of the infrequent locations where corroded reinforcement is evident contained 0.10 percent chlorides.

Effect of Sea Salt on pH:

The solubility of calcium hydroxide is 0.165 grams per 100 grams of water at 68°F. The solubility of the principal salts of sea water is about 200 times as great. When a limited quantity of water is available as a solvent the amount of calcium hydroxide that can be in solution is limited by its low solubility but the concentration of sea-salt can be very much greater if the salt is available. The pH of sea water is about 8 and the pH of a saturated solution of calcium hydroxide is about 13. The pH of a saturated solution of calcium hydroxide containing varying amounts of sea-salt will vary between these limits. Figure 3 contains a curve showing this relationship as determined experimentally using evaporated sea-salt from the water at the bridge site. The curve indicates a rapid reduction in pH with increasing sea-salt up to about 2 grams per 100 ml. of calcium hydroxide solution at which point the pH is about 10. Further additions of sea-salt decrease the pH at a reduced rate and for quantities in excess of 4 grams per 100 ml. the rate of change is relatively slight.

Neglecting for the moment the fact that the curve represents a saturated solution of pure calcium hydroxide, whereas other compounds would be dissolved in water in contact with

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portland cement, it is seen that theoretically a pH of 10 would result when the concentration of sea-salt in the water present in concrete amounts 2 grams per 100 ml. The data presented in the preceeding section show that sea-salt in excess of 0.45 percent (0.25 percent chlorides) was found frequently in the samples that were analysed. The absorption of concrete from the beams, caps and piles were shown in Table II to be about 7 percent which is equivalent to about 12 percent of the mortar. With 0.45 percent sea-salt its concentration in the water of saturated concrete would be about 3.7 grams per 100 ml. From the calcium hydroxide curve of Figure 3 the resulting pH would be about 9.5.

Figure 3 also contains a curve for which the solution is a water extract of freshly hydrated portland cement paste. The paste was made with Golden Gate cement (the brand used in the bridge) which contained about 1.10 percent alkalies (Na_2O and K_2O). The paste was mixed at normal consistency and was cured in the moist room for 7 days. It was then oven-dried and granulated. Water was added to bring the total quantity to 7-1/2 gallons per sack of original cement and the mixture was agitated continuously for 3 days and then filtered. Varying amounts of sea-salt and a small amount of hydrated cement were added to portions of the extract. pH values were determined at intervals up to 7 days at which time they were constant. Each sample was then analysed for chlorides and calcium hydroxide in solution. The data were used to construct the curve as shown in Figure 3. The data of the tests are given in Table XI at the end of the report. Only one series of tests was made and the shape of the resulting curve has not been verified.

It is reasonably certain that substantially all of the alkalies of the cement were present in the water extract. This result is believed to have been mainly responsible for the displacement of the cement extract curve relative to that for pure calcium hydroxide. It is to be expected that a similar curve representing a water extract of cement from the bridge at its present age would lie between the two curves of Figure 3 because of the loss of alkalies through leaching.

pH values of two moistened samples of pulverized mortar from the bridge were determined. Increments of sea-salt were added to sample R and the actual contents of chlorides were determined. The results are shown graphically in Figure 4. The plotted points are for moisture contents of 8 percent (about 75 percent saturation) and higher. 8 percent was the lowest moisture content at which pH could be measured with a glass electrode. The curves have been extrapolated into the region of lower moisture. Sample I presumably has lost more alkalies through leaching than has sample R and therefore has a lower pH for the same salt content.

The method of preparing the mortar samples for test was as follows. Water was added to the pulverized, oven-dried samples, with or without added sea salt and mixing was performed in a saturated atmosphere to avoid loss by evaporation. The

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samples were then sealed in chemically resistant glass bottles. pH readings were made after 2, 4 and 8 days and appeared to be stable at the latter date. Each recorded value was the average of several trials. The values used in plotting Figure 4 are the averages of the 4 and 8-day readings. Considerable difficulty was experienced in obtaining reproducible readings and smooth curves could not be fitted to include all of the plotted points. Nevertheless the trend is unmistakable and it is believed that the curves as drawn are fair representations of true conditions.

The data indicates the probability of pH values as low as 10 to 11 in unsaturated concrete containing sea-salts in the amounts found to be present in samples from the bridge. (Refer to Table X.)

This subject has been presented in considerable detail in order to establish the important function of sea-salt in lowering the pH of concrete.

SPACER BLOCKS

During construction, small, prefabricated mortar blocks were used to support the reinforcing steel above the forms at the bottom of beams. Four or five pairs of blocks were used per beam and they were placed under stirrups at approximately, but by no means exactly, equal intervals. During repair work it was noted that there was substantially no bond between the concrete and two or more faces of the blocks. As a result the blocks could be recovered quite easily and about 40 of them from four beams were obtained for examination and testing.

Dimensions of the blocks were found to vary considerably but the average size was about 2-1/2" square by 1-3/4" high.

The composition of the blocks is given in Table XII.

TABLE XII

Composition of Spacer Blocks

Mix Proportions	1 cement: 2.1 sand, by
Magnesia, MgO	5.1% by weight of cement
Sulfuric Anhydride, SO_3	1.68% by weight of cement
Chlorides (20 blocks tested)	0.053 minimum 0.585 maximum 0.175 average

Results of physical tests of the blocks are given in Table XIII.

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TABLE XIII

Physical Tests of Spacer Blocks

Absorption of oven-dried blocks in cold water, %			Unit Weight After soaking for 7 days lbs. per cu. ft.
1 day	2 days	7 days	
6.03	6.10	6.30	138.2
6.25	6.43	6.64	137.7
6.20	6.33	6.46	138.0
4.81	5.13	5.35	136.7
5.91	6.03	6.20	138.2
5.84	6.00	6.20	137.8

Compressive strength of sawed cubes after soaking in water for 7 days. Values in psi after correcting to height equal to twice the width.

5620
6480
6250
5200
5890

The data of Table XII show that the mortar of the spacer blocks has been infiltrated with sea-salt to about the same degree as the surrounding concrete. The content of magnesia suggests sulfate attack. The absorption and compressive strength data of Table XIII indicate that the mortar is of higher qualify than that of the surrounding concrete. The strength level while relatively high appears to be lower than would be expected in unaffected mortar of the same mix and age.

Lack of bond between concrete and spacer blocks suggests the existence of virtual cracks at these points and the attendant possibility that the reinforcing steel would be more vulnerable to attack by corrosion at these locations.

The concrete was removed full length from the bottom of four beams to expose fully the lower line of reinforcing steel. The position of each stirrup and spacer block was recorded. The degree of corrosion in each bar at each intersection with a stirrup and in the intervening space was recorded. The degree of corrosion was rated in accordance with the following scale:

None: Bar apparently unaffected since installation.

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Trace: Slightly more corrosion than was assumed to be present when concrete was placed.

Light: Distinct corrosion not built up to a lamellar condition

Medium: Corrosion product in visible layers

Severe: Heavy lamellar deposit, 3/32 inch or more in thickness.

Examinations and notations were made by Mr. M. W. Gewertz, Senior Bridge Engineer.

The data of these notes have been plotted in Figure 5. The condition in Span 387, Beam S-1 (upper right of figure) shows particularly well that corrosion is more severe in the vicinity of the spacer blocks and suggests that it may have started at these locations and later spread in one or both directions from these points. In other beams where corrosion is more advanced the relation to the spacer blocks is less clearly defined.

Taken as a whole, the frequency at which severe or medium corrosion has occurred within 1 foot of a spacer block is about 60 per cent greater than over the entire length of the bars.

In the four beams there were 18 pairs of spacer blocks each supporting stirrups which in turn supported four longitudinal bars. There were thus 72 points of "contact" between bars and spacer blocks. At 19 points the corrosion was at a low level at the block and for several feet each side. Of the remai ing 53 points the following relationship obtained:

Corrosion severe at or within 1 foot of spacer block	34 points or 64 percent
Corrosion medium and definitely higher at spacer block than in interval of several feet each side	7 points or 13 percent
Corrosion less at spacer block than in adjoining spaces	12 points or 23 percent

The evidence is therefore quite strong that the spacer bl introduced points of weakness with respect to the prevention of corrosion.

Chloride determinations of the blocks in two beams are indicated on the diagrams. Relatively low percentages of chlori were found in the blocks nearest the west end of the beams and corrosion in this vicinity was of low order. Over the remaining portion of the beams there appears to be little relationship between the percentage of chlorides and the degree of corrosion.

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The relationship discussed above has been confined to beams. It is reported that spacer blocks have been found occasionally in the piles but these members could not very well be subjected to a study of the nature of that made on the beams. As far as known spacer blocks were not used in the caps.

GENERAL OBSERVATIONS

During the course of the investigation a few observations of possible pertinence were made. In addition, information was obtained in conversations with engineers and workmen on the bridge.

Reinforcement in the beams and caps consisted of ovoid bars with the major axis horizontal. Corrosion appeared to be much heavier in the lower half of the bars. This condition may be connected with a possible minute void under the bar due to water gain.

The radius of bend of the stirrups was rather large with the result that the lower line of bars in the beams was crowded toward the center. Cover over the outer bars in a horizontal direction approached 4 inches whereas it was approximately 2 inches measured downward. The lower line of bars was not always straight and frequently the bars were separated from the stirrups by as much as 1/2-inch.

Concrete, in the beams particularly, appeared to be unusually damp with respect to prevailing weather conditions. It is believed that the bituminous coating tended to maintain a higher than normal level of moisture. It is reported that occasional blisters in the coating were filled with liquid.

In contrast with the lower portions of the beams the bottom face of the deck appears to be in remarkably good condition in view of the fact that the deck was cast integrally with the beams. There are no signs of leakage through the deck and efflorescence is at a minimum.

Workmen report that the upper ends of some piles for a distance of about a foot is unusually soft. It is possible that soft concrete is found in those piles that were extended on the job after they were driven.

DISCUSSION

The test results reported in the foregoing sections were obtained on specimens from but a few of the 1170 spans making up the concrete structure. Sampling certainly was inadequate to indicate more than a trend as to the condition of the bridge as a whole. With this limitation in mind the following

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are given as the major findings of the study.

The concrete placed in the beams and caps contained about 5.3 sacks of cement per cubic yard and consequently a water-cement ratio of about 7.5 gallons per sack was required. The American Concrete Institute (3) recommends not more than 5.0 gallons per sack for "moderate sections such as girders and beams, at the water line or in the range of spray from sea water, mild temperature rarely below freezing". The water-cement ratio used in the beams and caps was thus far above that now recognized as good practice. Absorption results substantiate the conclusion of a high water-cement ratio. Permeability to air is high. A few test specimens were sufficiently permeable to water to permit practically gravitational flow. In the transpiration test the flow of water in the liquid or vapor phase was exceptionally high thus indicating conditions leading to high absorption of salts. The compressive strength at present is low and is approximately the same as that of test cylinders at the age of 28 days. Since the concrete should have gained in strength during the years, a retrogression from a previous higher value is indicated. The content of magnesia and sulfuric anhydride is high and indicates that the concrete has suffered from sulfate attack. The present low strength bears out this assumption. The portland cement used, because of its high content of tricalcium aluminate, was of a type that is not resistant to sulfate attack. There appears to be a definite probability therefore, that the concrete will continue to deteriorate. This conclusion is reached notwithstanding the present outward appearance of the concrete and the relativly high pulse velocity as determined with the soniscope.

The concrete in the piling had a cement factor of about 7 sacks per cubic yard and a consequent water-cement ratio of about 5.7 gallons per sack. The American Concrete Institute (3) recommends not more than 4.5 gallons per sack for "reinforced piles in sea water, mild temperature, rarely below freezing". The water-cement ratio used in the piling, therefore exceeded that now recognized as good practice by a considerable margin. Absorption tests of cores from the piles were almost as high as that found in the beam and cap specimens. Permeability to air while distinctly lower than that of the caps is as high or higher than that of the beams. In the transpiration test the pile specimens, while distinctly better than those from beams and caps is nevertheless undesirably high. The compressive strength is now at about the same level as that of construction test cylinders at the age of 28 days. The same comments made with respect to sulfate attack and retrogression in the beams and girders apply to the piles as well.

Sea-salts have entered beams, caps and piles in amounts sufficient to lower the pH of the concrete to a level below that required for good protection of the reinforcing steel against corrosion. The presence of salt promotes corrosion in other ways as well.

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Uhlig(1) attributes increase in corrosion rate due to salt to two factors: (1) the electrolytic conductivity of the solution permits operation of anode and cathode areas further removed from each other, and (2) the anode and cathode reaction products tend to combine at a place removed from the iron surface and hence precipitate a less protective corrosion product layer.

Rosa, McCollum and Peters (6) state:

> "The addition of a small amount of salt (a fraction of 1 percent) to concrete has a two-fold effect, viz, it greatly increases the initial conductivity of the wet concrete thus allowing more current to flow and it also destroys the passive condition of the iron at ordinary temperatures, thus multiplying by many hundreds of times the rate of corrosion and consequent tendency of the concrete to crack.
>
> "Oxides of iron occupy 2.2 as much space as the metal.
>
> "The mechanical pressure developed at the iron anode by the corrosion of iron has been measured in a number of cases and has been found to reach values as high as 4700 pounds per square inch, a value more than sufficient to account for the phenomena of cracking that have been observed."

The preceeding statements were made by Rosa, McCollum and Peters in connection with a study of electrolysis in concrete in which impressed voltages were from 15 to 120. Nevertheless the findings should apply in principle to conditions at present assumed to exist at the San Mateo-Hayward Bridge, that is, through the operation of local cells only.

Precast spacer blocks of mortar were used to support reinforcing steel above the bottom of the forms for beams and probably for piles. In the beams there is in many cases a complete absence of bond between the blocks and the surrounding concrete. There is strong evidence that the blocks frequently were the starting point of corrosion in the lower line of reinforcing steel in the beams. The quality of mortar in the blocks is better than that in the surrounding concrete. It seems certain that corrosion would have been retarded markedly but not necessarily prevented entirely, had steps been taken to obtain a good bond between the blocks and the concrete.

It does not seem to be necessary to assume the existence of stray currents of sufficient voltage to cause electrolysis in order to account for the corrosion that has taken place.

In discussing the quality of concrete as placed there is no intention of reflecting adversely on the ability of the

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engineers who designed and constructed the bridge. Twenty-five years ago present-day guides to durable concrete had not been formulated. It was generally believed at that time that durability could be measured by strength alone. During construction, design strengths were obtained with rather wet concrete of low cement content. Undoubtedly the engineers would have been criti had they used richer mixes. At that time there was no informati available as to the effect of tricalcium aluminate in cement on resistance to sulfate attack. Sulfate-resisting cements such as Type II and Type V had not been developed.

UNANSWERED QUESTIONS

The scope of the investigations reported herein has been insufficient to provide answers to many questions that arise in connection with the past history and future service of the bridge Among them are the following:

(1) Is the character of the concrete that was sampled and tested sufficiently representative of the bridge as a whole?

(2) Is the concrete continuing to deteriorate as a result of sulfate action and, if so, what is its probable serviceable life?

(3) Why is the deck in so much better condition than the beams that were cast monolithically with it?

(4) Did the application of the bituminous coating retard or promote the development of corrosion? Would newly placed shotcrete be benefited by a similar coating?

(5) The quality of the shotcrete that was placed in prior years appears to have been good yet it was not effective in preventing further corrosion. The shotcrete now being applied does not in itself appear to be superior to the older material. Will the newly adopted practice of thoroughly cleaning the reinforcing steel before placing the shotcrete, by itself result in a permanent repair?

RECOMMENDATIONS

With one exception, definite recommendations as to future investigations do not appear to be warranted until the results of the second phase dealing with electrical aspects have been reported. The single recommendation made at this time is as foll

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A program should be initiated to study the course of terioration of the concrete due to sulfate attack. As visioned this program would involve the taking of cores at refully selected locations in the structure. The cores would tested to determine the present compressive strength of the rete. At periodic intervals of possibly a year similar es would be taken and tested. Concurrently with the core udies, pulse velocities at permanently marked locations would measured with the soniscope.

CONCLUSION

While all factors have not been thoroughly explored in is preliminary investigation it seems clear that the quality the concrete is primarily responsible for the observed stress. While it is difficult to afford perfect protection reinforcing steel in concrete exposed to sea water, there are y examples of greater durability. For example, 1200 precast einforced concrete pile jackets in Pier 17 of the San Francisco rbor installations have given 36 years of satisfactory rvice (7).

While the mechanism of corrosion is involved and is an lectro-chemical phenomenon, the relatively pervious concrete on e San Mateo Bridge permitted salt from wave action, spray or pillarity to penetrate to the reinforcing steel and undoubtedly s the primary factor in the subsequent cracking and spalling of e piles, caps and girders.

The pilot study was not sufficiently extensive to warrant prediction as to the future performance of the old structure or the repair work thus far completed.

ACKNOWLEDGEMENT

The Materials and Research Department is indebted to the taff of the State-owned Toll Bridge Section for information elative to the design and history of the project, for assistance securing samples for test and for helpful cooperation in all ases of the field investigation.

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TABLE VIII

Leaching and Carbonation

Core No. 7 (from a beam) was sawed into 1-inch Sections. The coarse aggregate was removed and the mortar pulverized. To each sample, weighing about 50 grams, 200 ml. of freshly boiled distilled water was added and the flask was stoppered. After standing 7 days with occasional agitation, aliquots were withdrawn and the following determinations made. Letters designate position in core, (a) being at the surface of the beam. Values are expressed as $Ca(OH)_2$ in parts per million.

	(a)	(b)	(c)	(d)	(e)	(f)
Alkalinity, phenolphthalein	1400	1500	1500	1550	1500	1550
Alkalinity, methyl orange	1475	1575	1575	1625	1575	1625
by precipitation as ocalate	800	1250	1250	1150	1200	1150
pH	12.2	12.2	12.2	12.2	12.2	12.2

Note: Solubility of $Ca(OH)_2$ in water at 21°C is 1600 ppm. It is probable that water in above tests did not become saturated in the time allowed. Results indicate the presence of alkalies as well as calcium hydroxide, with greater concentration of alkalies at the surface.

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TABLE XI

Effect of Dissolved Sea-Salt on pH of Saturated Calcium Hydroxide Solution and Extract of Hydrated Portland Cement

Calcium Hydroxide		Hydrated Cement		
Gms. of Sea-Salt* added to 100 ml. of Solution	pH	Gms. of Sea-Salt* added to 100 ml. of extract	Gms. of $Ca(OH)_2$ in 100 ml. of extract	pH
0	12.8	0	0.023	12.8
1.0	11.9	1.0	0.027	12.7
2.0	10.1	1.9	0.028	12.6
3.0	9.7	2.9	0.039	12.5
3.9	9.5	3.9	0.051	12.4
4.9	9.4	4.7	0.069	12.3
7.4	9.3	7.3	0.125	12.1
9.7	9.2	9.5	0.206	11.9
18.7	8.9	12.2	0.318	11.1
26.9	8.6	19.1	0.268	9.1
		27.2	0.174	8.6

* Based on analysis of solution for chlorides

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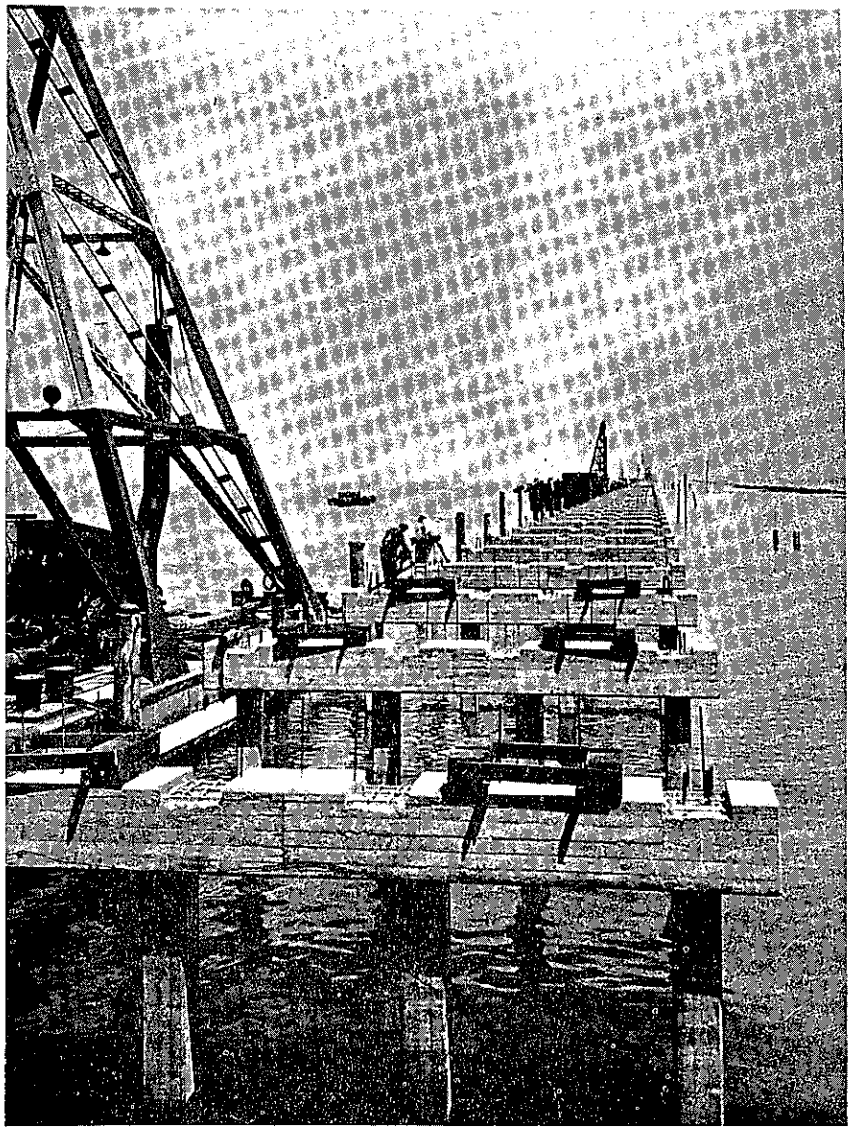

Figure A
San Mateo-Hayward Bridge During Construction

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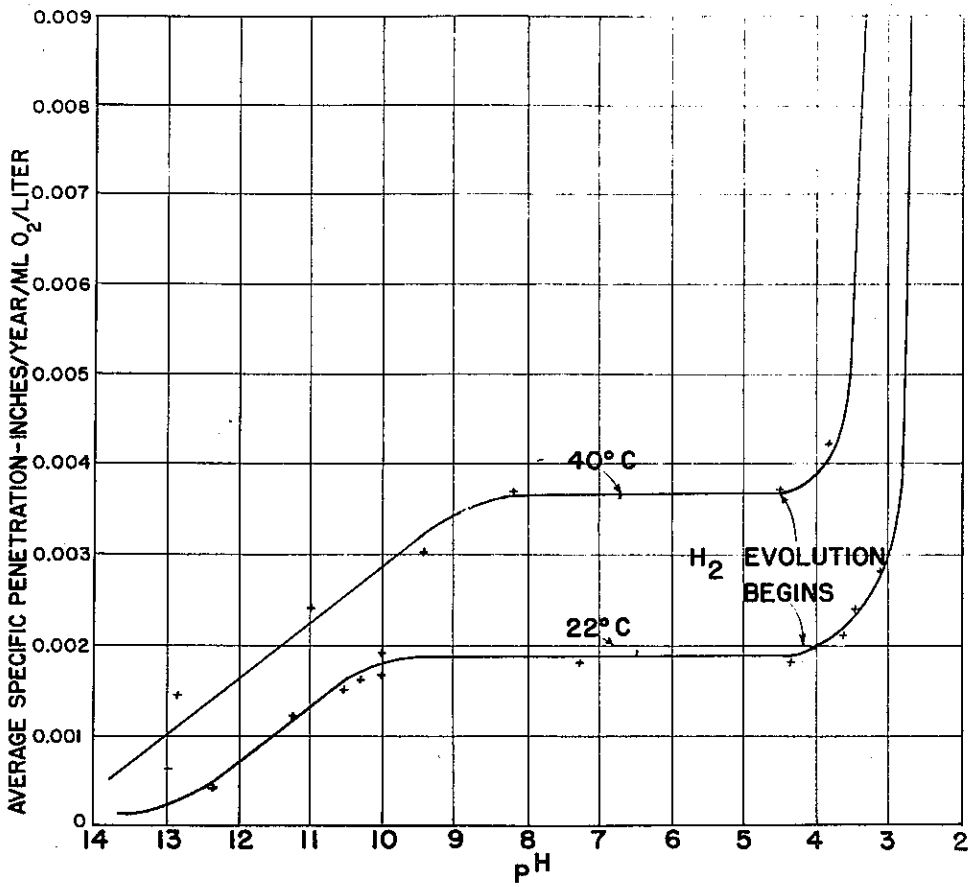

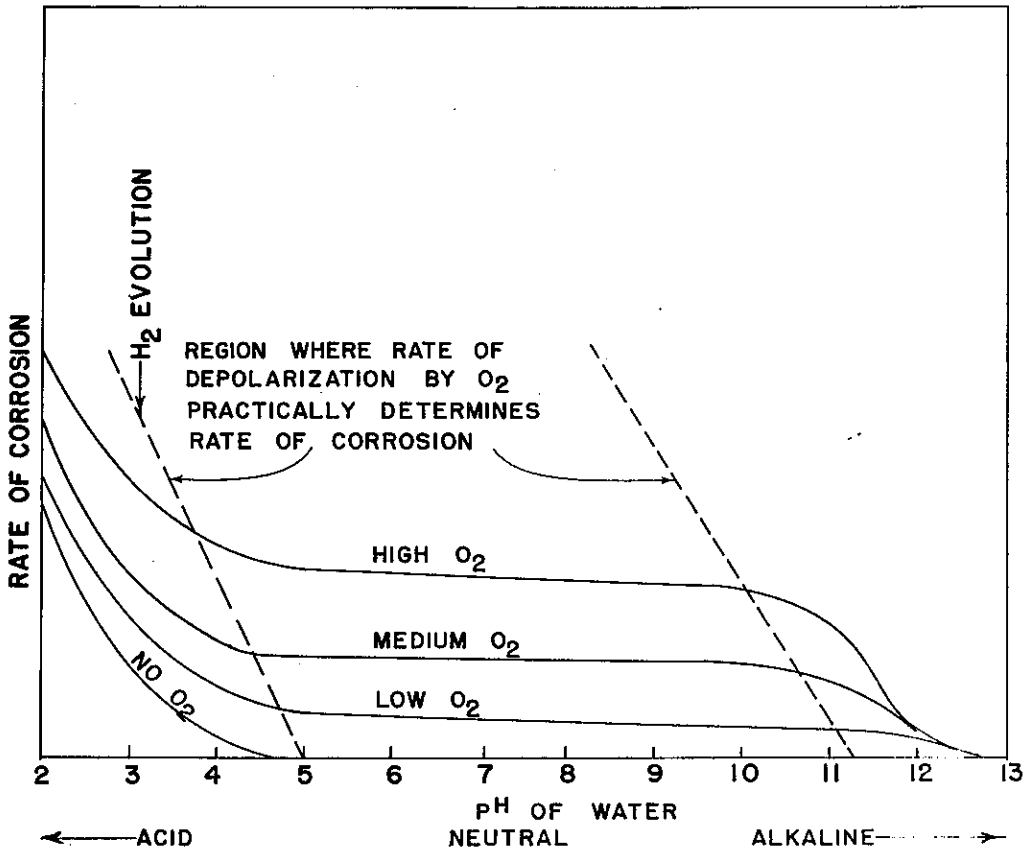

FIGURE I
EFFECT OF PH OF AQUEOUS SOLUTIONS
ON
RATE OF CORROSION ON STEEL

5006-R-54 (PART I)

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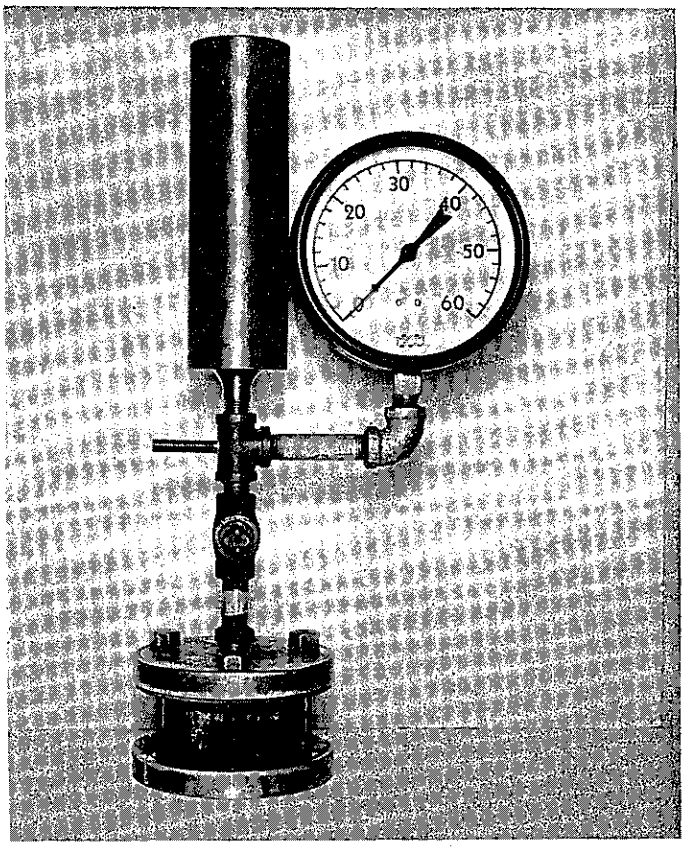
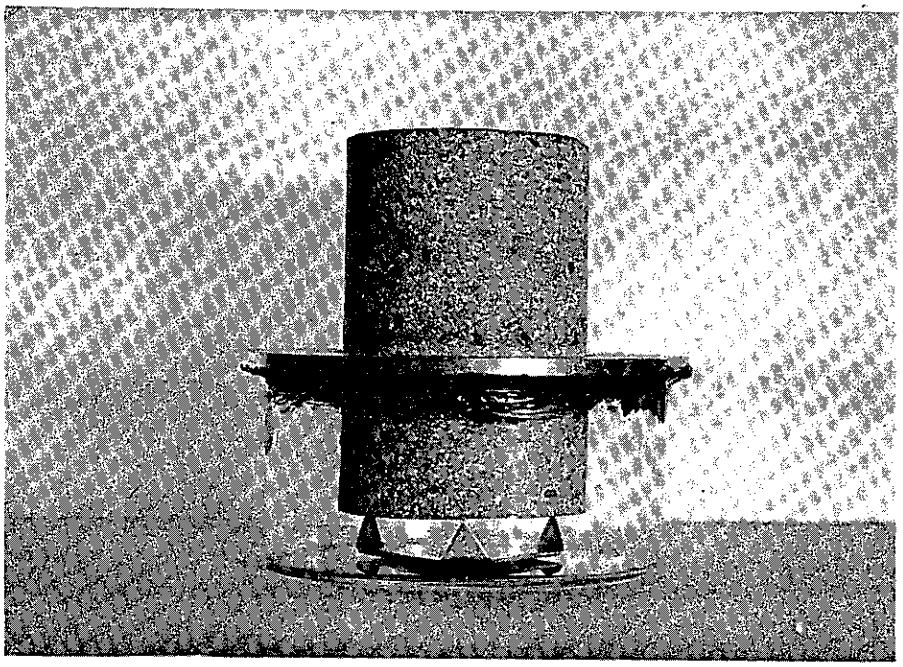


Air-Permeability Apparatus



Transpiration Test

Figure 2
5006 R 54 (Part 1)

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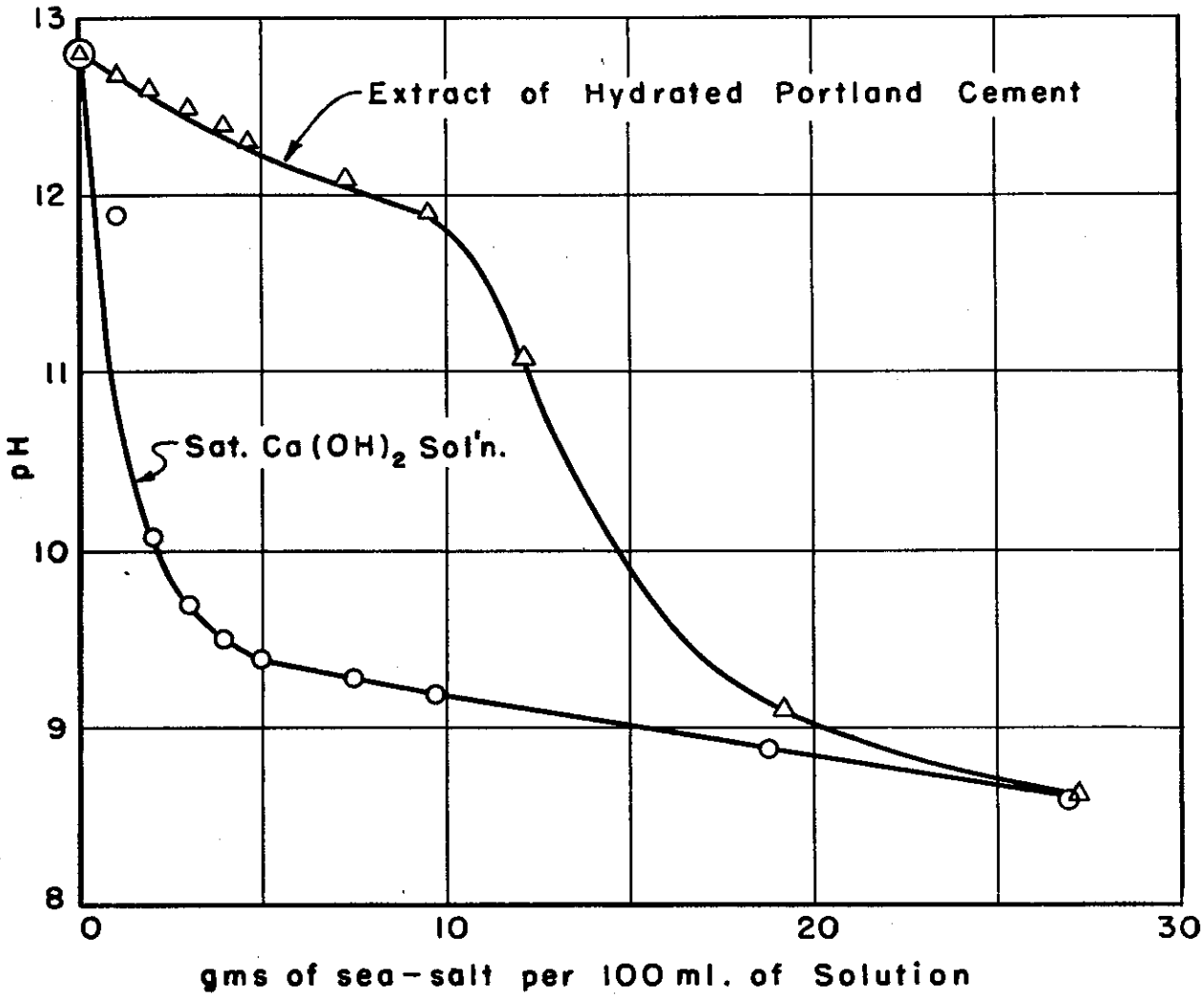

FIGURE 3.

Effect of Sea-Salt on pH of Calcium Hydroxide Solution and Hydrated Portland Cement Extract

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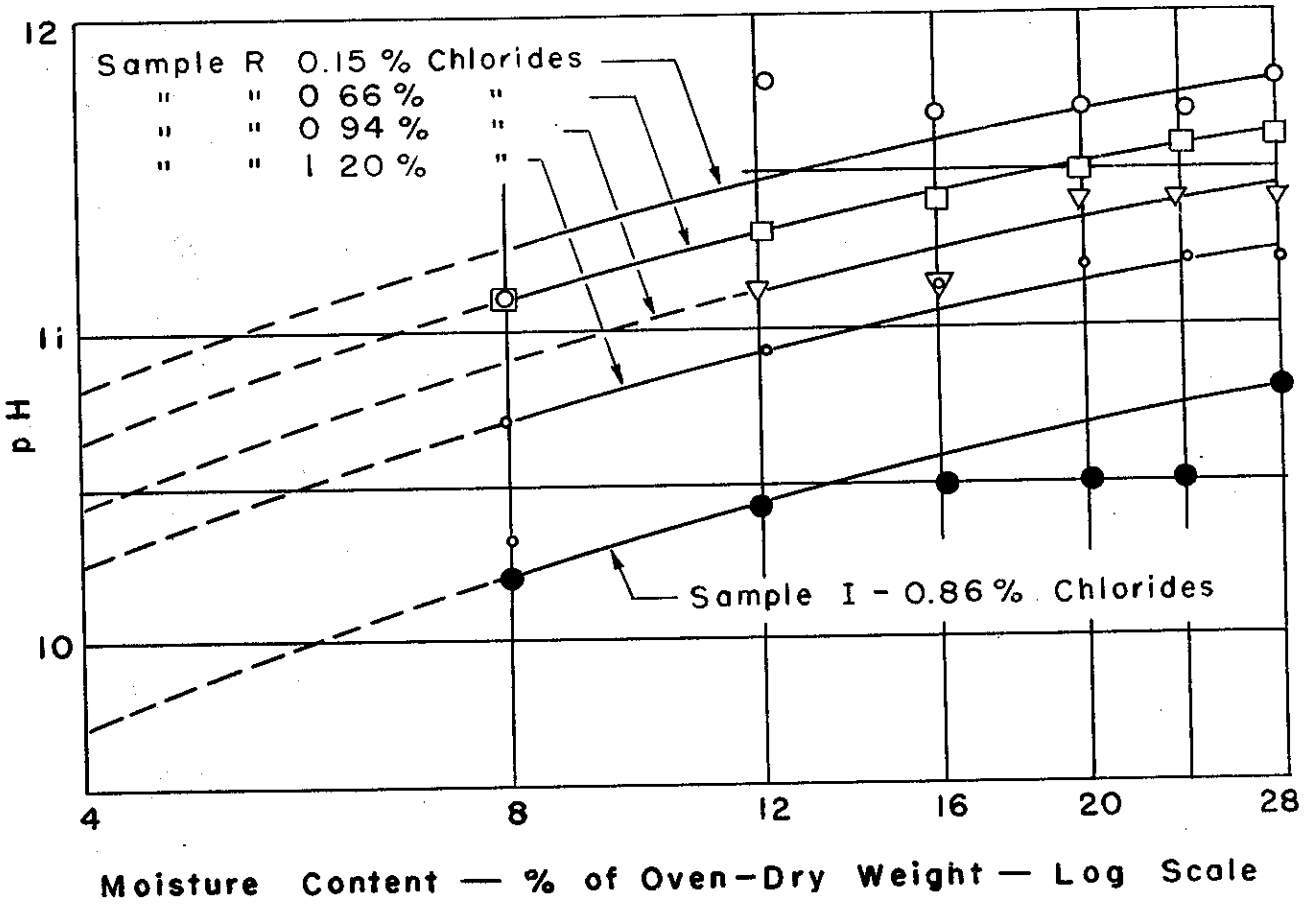

FIGURE 4.

EFFECT OF MOISTURE ON pH OF MORTAR CONTAINING SEA-SALT.

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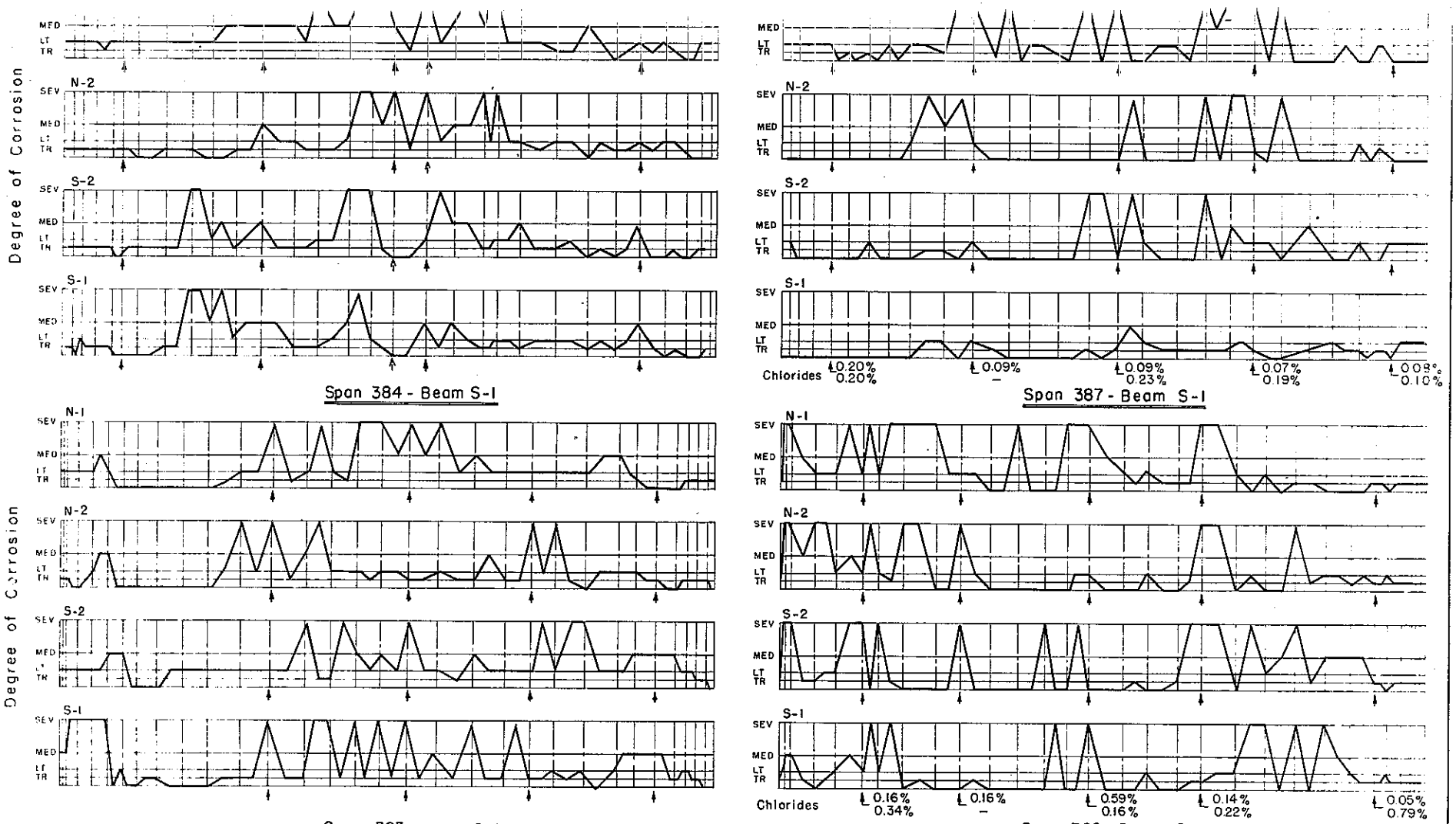


Note: Arrows indicate location of spacer blocks.

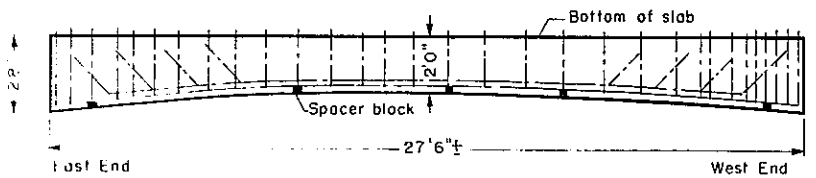
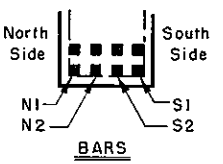


Diagram of Beam



BARS

FIGURE 5

DISTRIBUTION OF CORROSION ON LOWER BARS OF BEAMS

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