Environmental Influence on Corrosion of Reinforcing in Concrete Bridge Substructures

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Prompted by evidence of corrosion which occurred in the reinforcing steel used in 20 highway bridges in an arid desert within 10 years after construction, a survey was made of the condition of 239 bridges located throughout the State of California. The survey indicated that corrosion of the embedded steel was evident in varying amounts in approximately 28 percent of the bridges. These structures ranged in age up to 50 years.

It was found that bridge deterioration was related to chloride content of the soils or waters in the environment. An equation was derived for the percentage of structures that had deterioration for any concentration of chlorides in the immediate area.

Laboratory tests continuing over a period of $2\frac{1}{2}$ years indicated a relationship between concrete's cement and water contents and its ability to gain and lose water vapor in controlled environments. Using the variables that appear to influence the movement of water vapor through concrete, an equation was developed which gives the probable time to cracking of a reinforced concrete substructure. The equation is based on field observations and laboratory tests, and takes into account the effect of the variables of cement and mixing water content, the thickness of cover over the steel, and the chloride content of the environment.

•**HUNDREDS** of reinforced concrete bridges have been constructed over the past 50 years in the State of California. The majority of these structures have required a minimum of repair attributable to the corrosion of reinforcing steel. However, there are a number of bridges showing evidence of cracking due to steel corrosion.

In California the most serious example of costly maintenance has been the deterioration repair of the San Mateo-Hayward Bridge (1). Although the deterioration of this structure has received considerable attention from the California Division of Highways, the distress of numerous reinforced concrete structures exposed to marine environments is not unusual (2). In a 1917 report (2) of the inspection of 146 structures on the east and west coasts of the United States, it was observed that "concrete can be used successfully in sea water, but the price of success is eternal care." This statement was based on existing technology as applied to a concrete coverage of 1 or 2 in. over the reinforcing steel.

Paper sponsored by Committee on Metals in Highway Structures.

In addition to the 1917 inspection of American-made structures, an inspection by the Committee of the Institution of Civil Engineers on structures in British waters in 1920 also revealed deterioration of numerous reinforced concrete structures (3). More recent reports (4, 5, 6) have shown that distress of reinforced concrete structures in marine environments continues. On the basis of these reports and the experience in California, it appears that due to corrosion of steel the cost of maintaining these structures may be expected to be extraordinary.

Previously, the investigation of the corrosion of reinforcing steel by the Materials and Research Department of the California Division of Highways has been primarily directed toward determining the effect of marine environments on the durability of reinforced concrete. However, in 1959, an investigation (7) was made of the distress of a 9-year-old highway bridge constructed in the arid Colorado Desert of California. It was found that this structure had the same evidence of distress as the San Mateo-Hayward Bridge (1) across the San Francisco Bay. On this same State highway, 66 reinforced concrete bridges were constructed during the years 1950-51, and 20 of the 66 bridges (about 30 percent) were found to have corroding reinforcing steel. Data collected during this investigation showed that the high concentration of salts in the soil (flow is usually not present at these bridge sites except once or twice a year) was responsible for the observed distress of the structures. Bridges in soils of high salt concentrations (up to 41,000 ppm of chlorides) showed distress, whereas those in low salt concentration (less than 200 ppm) were in a satisfactory condition.

Because of these findings on corrosion of reinforcing steel in an environment other than marine, a statewide investigation was made to determine the scope of this problem. This paper presents the results of this investigation.

CONDITION OF BRIDGES

The types of environments in which bridges in California were inspected for possible corrosion of reinforcing steel were (a) coastal, (b) valley, (c) desert, and (d) mountain. The 239 bridges inspected varied in age up to 50 years. The survey of the structures was visual and was concerned with determining the incidence of concrete cracking parallel to the reinforcing steel or rust stains on the surface of the concrete. These criteria were considered indicative of the corrosion of reinforcing steel. Typical examples of these observed conditions are shown in Figures 1 and 2. Figure 1 shows the cracking of reinforced concrete piles in a 10-year-old bridge exposed to tidal water near San Diego, Calif.; Figure 2 shows an area of advanced concrete spalling and the exposed reinforcing steel on the San Francisco-Oakland Bay Bridge.

Of the 239 bridges inspected, 66 showed evidence of corrosion of the reinforcing steel in some member of the structure. For the purpose of this investigation, only the reinforced piling, piers, or walls were considered to be directly affected by the variables found there. Distress observed in the rail, beams, etc., of the bridge could conceivably be influenced by air-borne salts in the atmosphere or by other atmospheric variables. Distress in these members was therefore considered outside the scope of this investigation. Based on this criterion, 37 structures showed visual evidence of corrosion of reinforcing steel in a pile or wall that was in direct contact with the natural soils or waters. The 29 structures that had visual evidence of deterioration in members other than the substructure were not used in the analysis that follows. Also, 5 structures which had less than 4 years of service were not used in the analysis.

In addition to the visual inspection of the condition of the structures, the following environmental data were collected for a possible correlation to the condition of the structures: (a) pH or hydrogen-ion concentration, (b) sulfates as SO_4, (c) chlorides as Cl, contained in the soils and the natural drainage waters, and (d) the electrical resistance of the soils and drainage waters.

CONCRETE DISTRESS

Chlorides

In the analysis of concrete distress due to the corrosion of reinforcing steel, it was not possible to determine when the crack in the concrete occurred. The only fact known

Figure 1. Vertical concrete cracking of 10-yr-old reinforced concrete bridge piles in tidal water near San Diego, Calif., 1962.

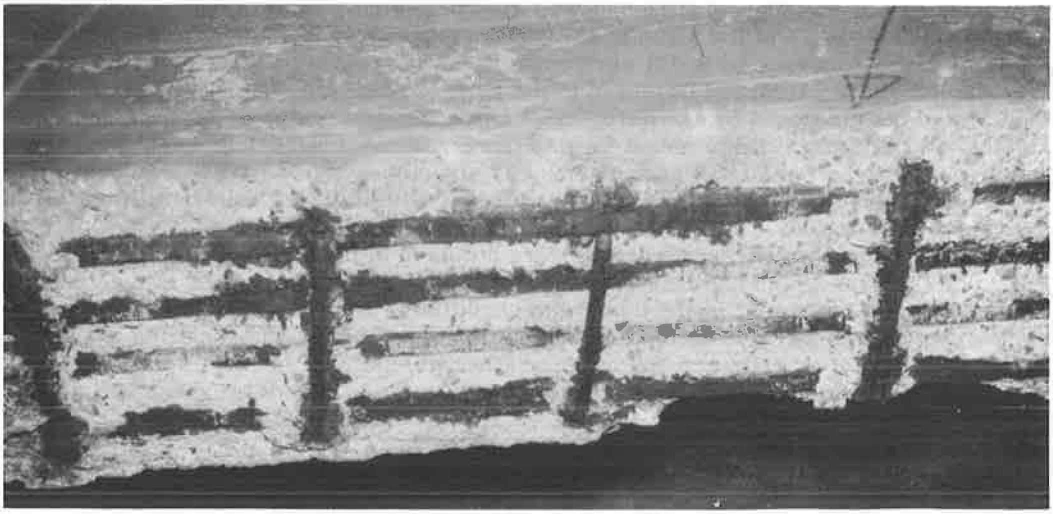


Figure 2. Spalling of concrete in beam caused by corrosion of reinforcing steel, San Francisco-Oakland Bay Bridge, 1962.

about the distress was that at the time of the inspection of a structure a crack was or was not present. Therefore, the data were analyzed by mathematically grouping the bridges into age vs deterioration and also vs the environmental factors.

Figure 3 is an example of the method used for plotting the "raw" data. The structures were first segregated into average age groups of 10, 20, 30, and 40 years of service. Then a plot was made of the condition of the structures at the particular

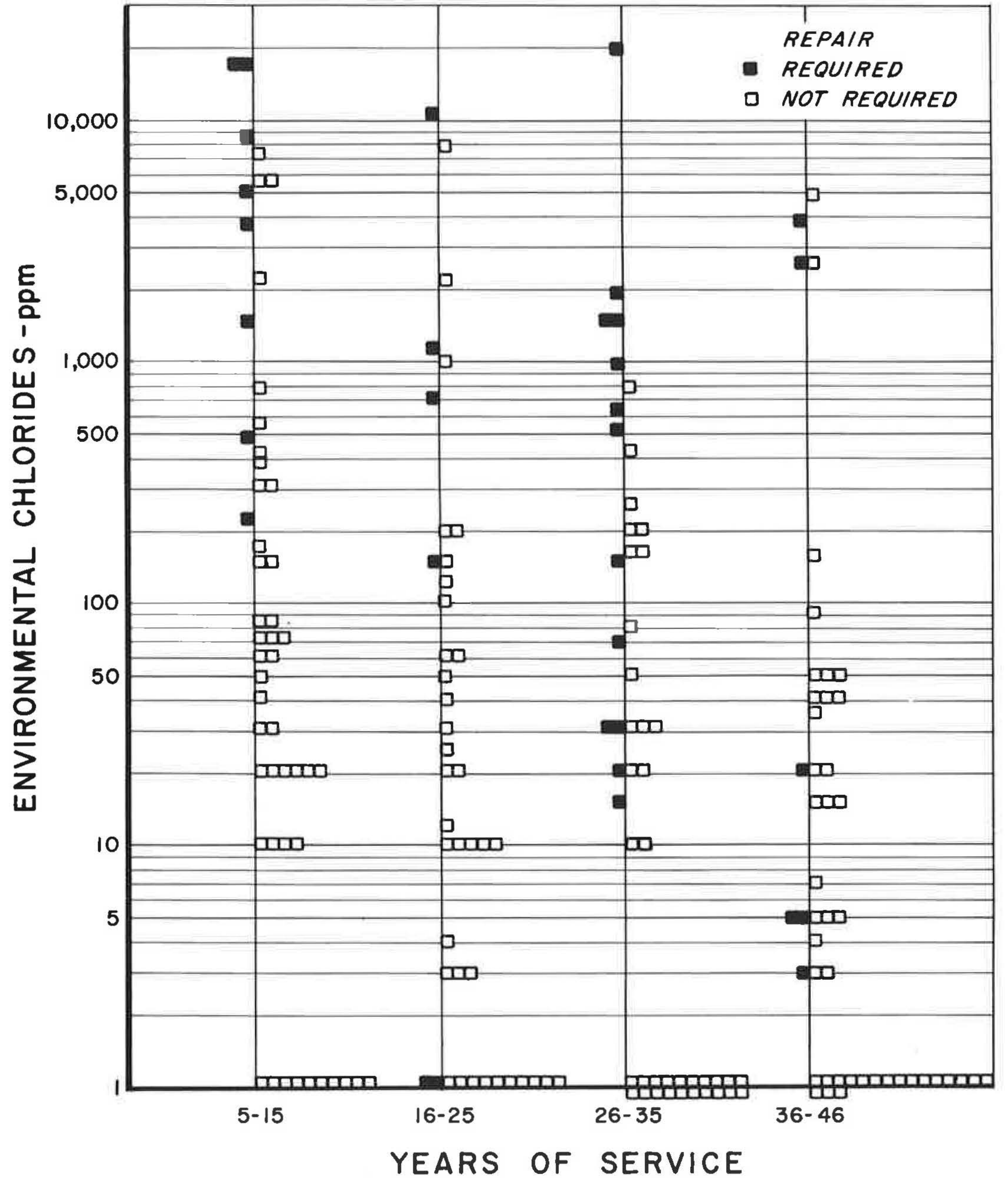


Figure 3. Influence of chlorides on deterioration of reinforced concrete bridge piles or walls.

chloride concentration of the environment. The figure shows that there is a relationship between the chloride concentration of the environment and the distribution of the condition of structures.

There were 29 structures that had visual evidence of deterioration in other than substructure members, 5 nondeteriorated structures with less than 4 years of service, 15 bridge locations in which a chemical analysis of the soil was not obtained, and 4 structures in which deterioration was observed after the initial investigation; these preceding data were not included as a part of the original data reduction. However, with judgment, some of the preceding data were included in Figure 10, discussed later.

The data (Fig. 3) were then put into distribution curves of the percentage of structures deteriorated at each of the four time increments. They were then mathematically analyzed to determine the yearly rate at which the structures would deteriorate in each chloride concentration. The result of the analysis is shown in Figure 4.

The correlation equation is

$$R_y = 0.165\ K^{0.42} \tag{1}$$

in which

R_y = percent of total structures showing distress per year; and

K = chloride concentration in parts per million found in either the soil or water that is in physical contact with the substructure.

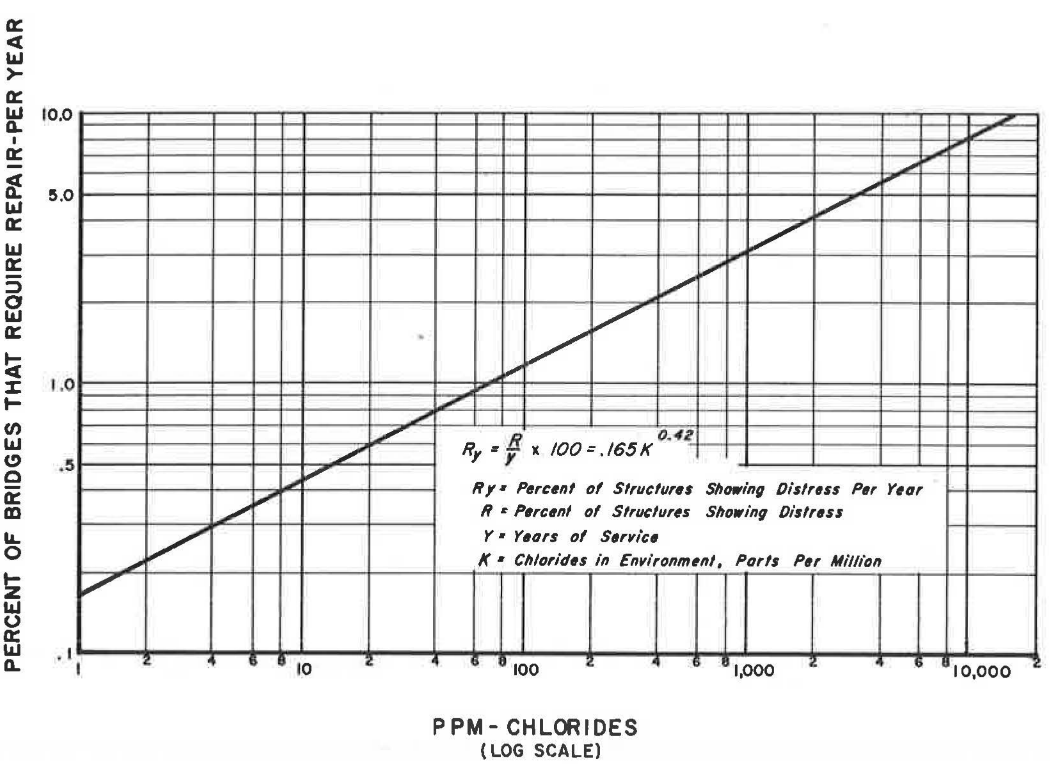


Figure 4. Correlation of chloride concentration and percent per year of bridges requiring repair.

After the equation was derived, it was tested for its ability to duplicate the original distribution curve of the condition of the bridges.

The method of least squares showed that there was a correlation coefficient of 0.957 and the standard error of estimate was 7.08 percent. This indicated that the results computed by use of the derived equation were within 7.08 percent of the actual percentages of structures found to be in distress when inspected. The level of significance of the correlation is greater than 0.001.

The data represented by Eq. 1 are typical of a distribution curve of the observed condition of a number of bridges. Analyzing the performance of a single structure would require a thorough investigation of all variables that might cause a deviation from the mean. Without doubt, there are variations in the relative protective value of concrete made throughout the years as a result of differences in environment, workmanship, curing, etc. For example, these data are for visible sections of a bridge substructure. The incidence of deterioration below ground or under water was not considered.

The concrete normally used for the construction of bridge piles consisted of 6 sacks of cement per cubic yard of concrete, and the design provided 2 in. of concrete cover over the reinforcing steel.

Sulfates

As previously stated, the sulfate (SO_4) contents of the soil and water, if any, at the bridge sites were determined and mathematically computed in the same manner as the chloride contents.

The analysis indicated that there was correlation between the sulfate content in the environment and the percentage of structures showing distress. However, it was generally observed that, when a high sulfate content was found in the environment, there was generally a high chloride content as well.

This observation suggested that there could be an interdependence between the chloride and sulfate concentration and the deterioration rate of the structures. To determine the possible interdependence of the chlorides and sulfate concentration and bridge distress in the environment, the data obtained for sulfates were analyzed for a correlation to bridge conditions in the same manner as for chlorides. Then the quantity of sulfates was plotted against the rate of bridge deterioration at each age group. At each of these same points of rate of bridge deterioration and age groups, the concentration of chlorides and sulfates was tabulated. The results of this comparison (Fig. 5) indicate that the more imminent cause of reinforcing steel corrosion is the chlorides present in the environment.

CONCRETE VARIABLES

Mixes

In addition to the study of the correlation between the chloride concentration in the environment and the condition of the bridges, studies were made of the vapor transmission characteristics of various types of concrete.

The corrosion of steel in concrete appears to be caused by the deposition of salts adjacent to the steel as a result of moisture movement. Therefore, the influence of the variables in mixtures was compared to the relative ability of the concrete to gain and lose moisture. The concrete test blocks were alternately exposed for periods of about four months (a) in a fog room at 73.4 F and 100 percent relative humidity, and (b) to drying at 73.4 F and 50 percent relative humidity. It was observed that the 3- by 3- by 11.25-in. concrete blocks would approach moisture equilibrium after three to four months of exposure.

In all cases, three concrete specimens were made with the following variables: (a) aggregate having 1.5, 4, and 10 percent absorption by weight, (b) equivalent of 4, 6, and 8 sacks of Type II cement per cubic yard, and (c) slump of 2, 4, and 6 in. The maximum aggregate size was 100 percent passing the ¾-in. screen. In addition, neat cement bars of 1 by 1 by 11.25 in. were cast with the equivalent of about 31, 32, 34, and 35 sacks of cement per cubic yard and the resulting neat cement flows of these mixes were 10, 37, 56, and 56, respectively.

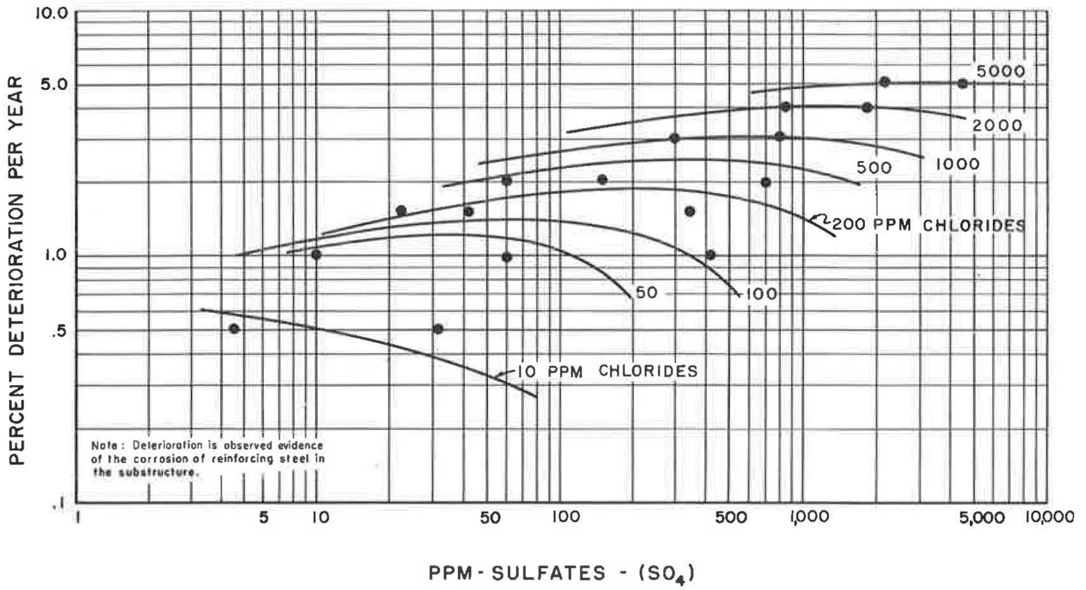


Figure 5. Influence of sulfate and chloride concentration on bridge deterioration.

All the concrete blocks and neat cement bars were cured for seven days in the fog room and then placed in the dry room for the beginning of the first dry cycle. The following data for the concrete blocks were obtained after approximately 2½ years of alternately exposing the concrete to the fog and drying room atmospheres. The study of the concrete blocks is continuing.

Water Voids

Various methods have been used to express the quantity of water gained and lost by concrete (8); however, this investigation was primarily concerned with comparing the gain and loss of moisture by various concrete mixtures under conditions comparable to a natural environment. Also, the measurement of the gain and loss of moisture was to be on a continuing basis in order to study the variable of continued concrete hydration; therefore, none of the concrete specimens has as yet been subjected to oven drying.

For this reason the term "water voids" has been used to describe the volume of water absorbed and evaporated by concrete that is subjected at 73.4 F to the alternate conditions of 50 and 100 percent relative humidity. None of the concrete specimens was subjected to other than the described environmental conditions except for the rare occasions when it was necessary to adjust the environmental rooms mechanically.

The weight of moisture gain and loss in the concrete specimens was measured to the nearest one gram during each cycle of exposure at time intervals of 1, 2, 4, 8, 16, etc., days. This was continued until the data indicated it would take more than one month of exposure for the concrete to have a weight gain or loss of one gram. Generally, this condition of moisture stability was obtained within approximately 128 days, or about four months of exposure.

After approximately one year of the alternate exposure of the concrete to fog wetting and drying, it was observed that the weight of water gained and lost for each triplicate set of concrete blocks was approaching a reproducible value. Therefore, the measurements of moisture movement considered significant in this study are for a period of exposure of the specimens that begins after the concrete has aged approximately one year. Concrete older than one year was termed "mature."

It has been previously mentioned that there were three types of aggregate used in the test with absorptions of 1.5, 4, and 10 percent by weight. The use of this range of aggregates resulted in concrete that varied in weight from about 150 to 110 pcf. Therefore, the measured weight of water gained and lost by the concrete specimens was compared on a volume basis to eliminate misleading data on variations that could occur as a result of extremes in concrete densities.

Figure 6 shows the relationship between the total water used in making the concrete and the volume of water that would be absorbed and evaporated during the environmental exposures of the specimens.

The derived water void equation shows that the relative volume of water gained and lost by the concrete in this test was a function of the total water used in the concrete and the quantity of cement. It was found that either an increase in the added mixing water or an excess contained in the aggregate increased the volume of water that transpired through the concrete for a given cement content. Conversely, for a given total mixing water content, an increase in cement content reduced the quantity of water that was absorbed or evaporated by the concrete under these test conditions.

The equation that shows the measured gain and loss of moisture in mature concrete is

$$V_w = \frac{W}{V} \times 100 = \frac{0.85 W_m^{1.17}}{C^{0.717}} \tag{2}$$

in which

V_w = water voids in percent of concrete volume as measured by loss in drying at 50 percent relative humidity and gain by exposure in fog room;

W = volume of water gained and lost;

V = volume of concrete = AS (area × depth);

W_m = total water contained in concrete mix as percent of concrete volume; and

C = sacks of cement per cubic yard of concrete.

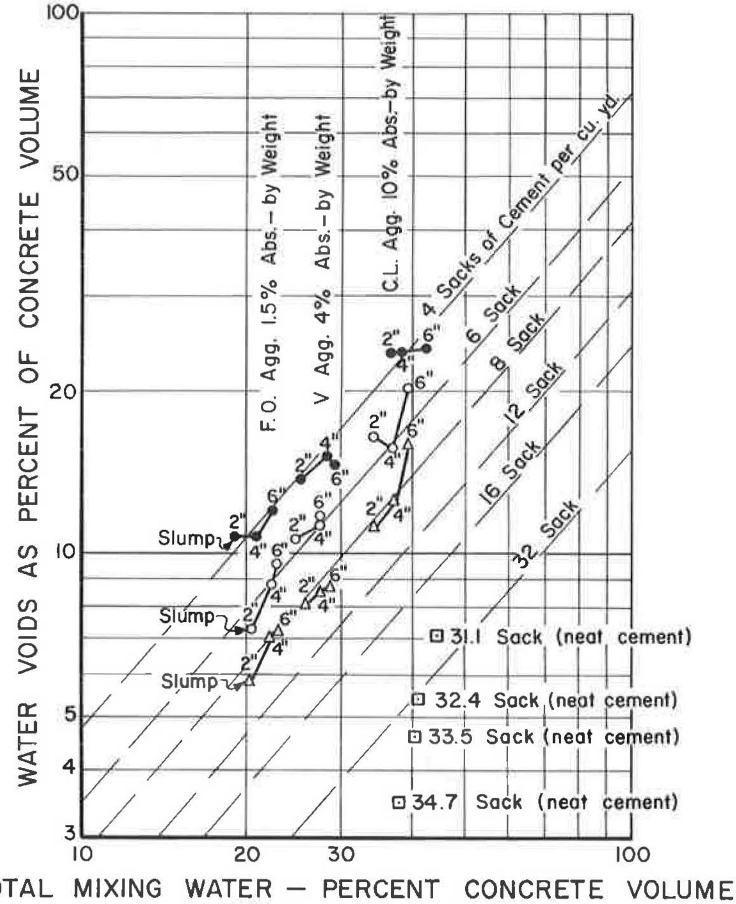
During the first year of measuring the water voids or the volume of moisture that transpired through the concrete test specimens, it was observed (Figs. 7 and 8) that the quantity of moisture movement would not be reasonably duplicated by each subsequent cycle of exposure. Further, under the test conditions it was approximately 200 days after the concrete specimens were cast before a maximum total movement of water would occur. This maximum movement appeared to occur when the blocks were placed in their first exposure to the fog room. The concrete made with the aggregate having 10 percent absorption was the exception.

The variations in the volume of moisture gained and lost during the first year of exposure appeared somewhat disconcerting when the data was analyzed; therefore, these variations were given additional mathematical study.

Figure 7 shows that the cement content and time are the variables responsible for the early differences in the volume of moisture absorption and loss in the concrete specimens. The data show that, with an increasing cement content, the volume of water movement decreases to a minimum value in proportion to the cement content. For instance, the concrete blocks that were made with the equivalent of 8 sacks of cement per cubic yard reduced about 35 percent from their maximum volume of moisture movement, whereas the 4-sack mix reduced about 18 percent.

The aggregates did not appear to exert a major influence on the change in the volume of moisture movement. This fact appears to be shown by Figure 8.

These data indicate that the absorptive and the evaporation characteristics of concrete are highly variable at early ages, and also vary with the cement content and the total water content of the mix.



$$V_w = \frac{0.85\,(W_m)^{1.17}}{C^{0.717}}$$

Figure 6. Water voids (computed from volume of moisture gained and lost during prolonged exposure to 73.4 F at 50 and 100 percent relative humidity, respectively) vs mixing water (including that in aggregate) in mature concrete (aged over 1 yr).

Rate of Moisture Movement

During the measurements of the gain and loss of moisture of the concrete blocks, the periodic quantity of water movement was found not to be constant but to decrease with the time of exposure. This observation was not considered unusual inasmuch as there is a limited quantity of moisture that would be available for evaporation. It was also obvious that when moisture had evaporated from the surface of a block, it would be readily apparent by a measurement of weight loss. However, the moisture that migrated from the center of the block toward the surface would not be detected by a weight difference until it evaporated into the atmosphere. Therefore, the rate of moisture change should vary as the distance that the water would be required to travel.

On an empirical basis, calculations were made on the rate and depth of moisture loss with the following considerations: The quantity of moisture that a concrete block contained was known by continuous weight measurements. It was assumed that the moisture was evenly distributed throughout the concrete mass at the end of each exposure period, and in all cases, could be computed for any point within the concrete

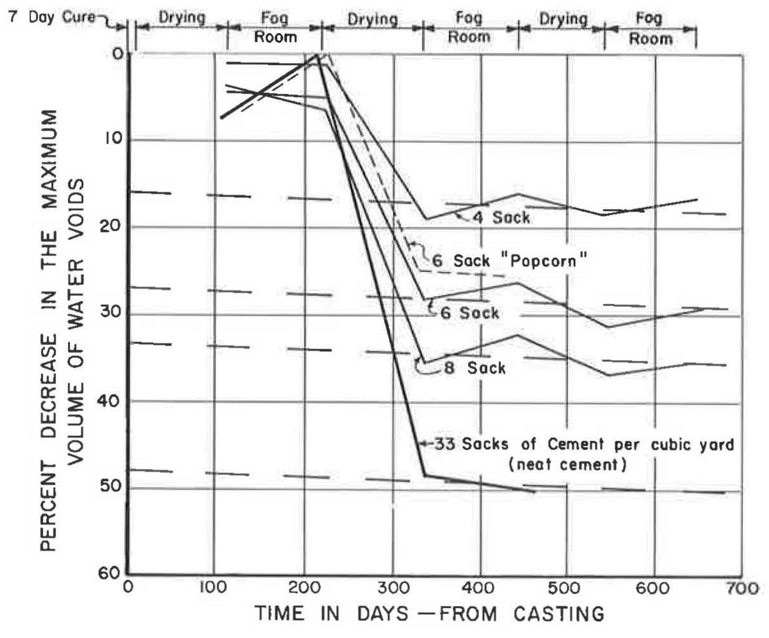


Figure 7. Influence of cement content on volume change of water voids-aggregate constant. Water voids computed from measured volume of moisture gained and lost during indicated exposure; concrete not oven dried; drying and fog room at 73.4 F, and 50 and 100 percent relative humidity, respectively.

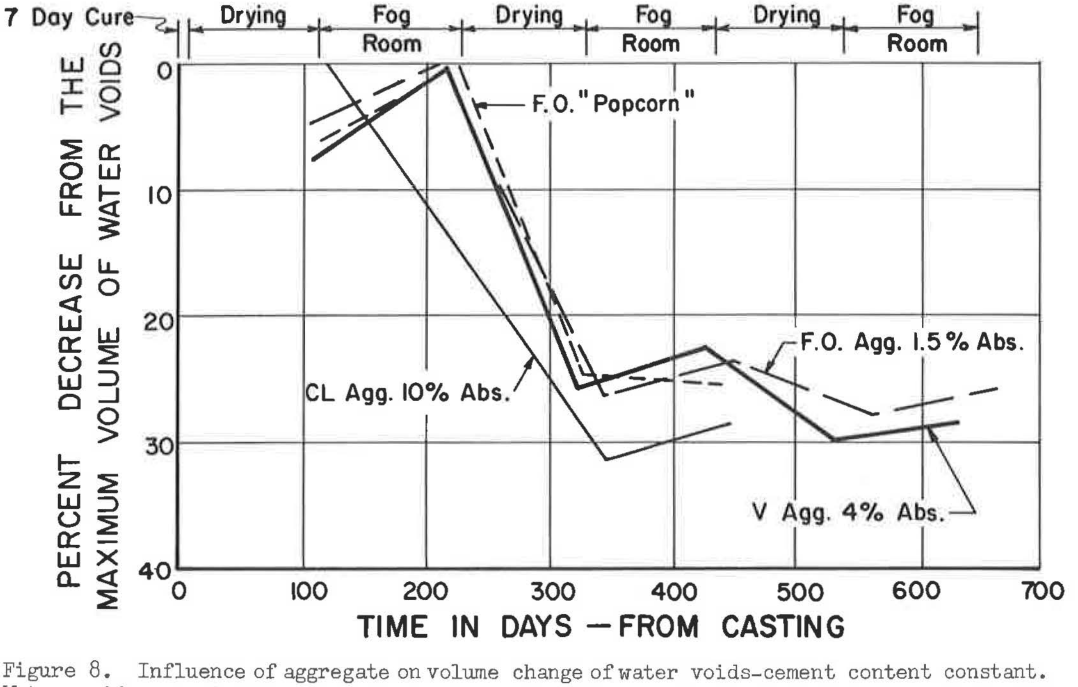


Figure 8. Influence of aggregate on volume change of water voids-cement content constant. Water voids computed from measured volume of moisture gained and lost during indicated exposure; concrete not oven dried; drying and fog room at 73.4 F, and 50 and 100 percent relative humidity, respectively.

on the basis of relative volume. Therefore, when a concrete block was in a saturated condition and lost a measured quantity of water, the volume of concrete that had contained this weight of water would be a function of the absorption of the concrete. Because the concrete blocks had constant dimensions, the depth below the surface of the block was computed, which would give the necessary volume of concrete that could hold the measured loss of water. Calculations of this type were used to determine the necessary depth below the concrete surface that would account for the measured gain and loss of water. It is readily apparent that this method of computation assumes that a portion of the concrete is relatively dry, whereas another portion may be saturated. The reality of this assumption is highly speculative, as would be other assumptions on the relative amount of moisture contained at various points within the concrete mass.

An analysis of the data indicated that there were two predominant variables which influenced the calculated rate of loss of moisture during the exposure in the dry room at 73.4 F and 50 percent relative humidity: (a) the cement content and (b) the assumed volume of the concrete that contained the evaporable water. A plot of these data is given by

$$t_y = 10^{0.0442C} \, S_i^{2.22} \, 0.086 \tag{3}$$

in which

t_y = time in years to dry to equilibrium with an environment of 73.4 F and 50 percent relative humidity;

C = sacks of cement per cubic yard of concrete; and

S_i = depth below surface in inches.

Analysis of the data indicated that the absorptive characteristics of the aggregate per se were not a predominant influence on the calculated rate of moisture evaporation.

The calculated rate of moisture movement through the concrete when exposed to the fog room environment of 73.4 F and 100 percent relative humidity implied that the concrete mix variables were not primary test variables. For a calculated depth of approximately 75 percent of the half-depth of the concrete, all specimens absorbed moisture at a rate of about 0.2 in. per day, then reduced in velocity at greater depths.

The results of the absorption test of the concrete in the fog room appear to be inconclusive in determining the influence of the concrete variables on the calculated rate of absorption. Although the blocks were exposed in a fog room of controlled temperature and humidity, the fog dispersion within the room was not uniform. For this reason the concrete blocks were moved about the room to average out the influence of differences of fog dispersion. It is possible that the variations of the fog room environment are greater than the influence of the concrete mixes. The data indicate that the rate of moisture absorption of the concrete in this test was greater than evaporation rate. Therefore, the evaporative characteristics of the concrete appear to be the controlling variable in the transpiration of moisture through concrete.

Relation of Laboratory and Field Studies

Field exposure tests showed that the protection of reinforcing steel by concrete is a variable that depends on the type of cement, water-cement ratio, admixtures, and thickness of concrete cover over the reinforcement (9, 10). Thus far, this investigation has not directly considered the influence of concrete variables on preventing or inhibiting the corrosion of reinforcing steel. However, it is probable that the durability of reinforced concrete will be a function of at least two of the many variables: (a) the absorption, and (b) the rate of moisture movement through the concrete.

The test methods used in the study precluded direct measure of the relationship of the saturated surface-dry to oven-dry method for measuring concrete absorption of water. However, the quantity of absorbed water found in this study could be related to the amount determined in the oven-drying method.

The concrete variables observed were related to the performance of the concrete in an atmospheric environment. It is believed that the behavior of concrete in such an environment is similar to the performance of concrete exposed near the ground or water line. This assumption is based on the measured differentials in the chloride content of concrete that has been exposed to a marine environment. An earlier study (1) showed that the chloride content decreases with depth below the surface. Therefore, with a given chlorinity of the environment, the observed differences in salt content of the concrete are assumed to be caused by wetting and drying.

With other environmental conditions being equal, it appears that the quantity of chlorides deposited within various concrete mixes could be directly proportional to the rate at which water could move from within the concrete to the atmosphere. This could be given by

$$Q = T_y W_v \tag{4}$$

in which

Q = quantity of water evaporated per year;
T_y = rate or cycles of evaporation per year; and
W_v = volume of water evaporated per cycle.

As previously stated, Eq. 3 gives the time in years when concrete would dry to equilibrium to a calculated depth. To obtain the rate or cycles of drying per year, it is necessary to take the reciprocal of that equation; thus, the rate per year of drying is

$$T_y = \frac{1}{10^{0.0442C} \; S_i^{2.22} \; 0.086} \tag{5}$$

The volume of water as a percent of concrete volume that would be evaporated from mature concrete is given by Eq. 2. Combining Eq. 2 with Eq. 4 gives

$$Q_y = T_y \; V_w \; V \tag{6}$$

in which

Q_y = quantity of water evaporated per year in cubic inches;
T_y = rate or cycles of drying per year;
V_w = water voids as percent of concrete volume; and
V = volume of concrete in cubic inches.

Substitution of Eqs. 2 and 5 in Eq. 6 gives

$$Q_y = \frac{0.0988 W_m^{1.17} \; A}{10^{0.0442C} \; C^{0.717} \; S_i^{1.22}} \tag{7}$$

in which

Q_y = cubic inches of water evaporated per year at 50 percent relative humidity and 734 F; and
A = cross-sectional area in square inches.

Construction records indicate that the over-all average design of the concrete mix for reinforced concrete piles for the past 50 years could be $C = 6$ sacks of cement per cubic yard of concrete, $W_m = 22$ percent by volume, total mixing water, and $S_i = 2$ in. of concrete cover over the reinforcing steel. Using these basic figures, the yearly quantity of water evaporated from an average concrete pile that is saturated with water and exposed to 73.4 F and 50 percent relative humidity could be, using Eq. 7, 0.2356A.

Eq. 1 gave the total yearly percentage of structures found in the field to have corrosion of the reinforcing steel. On a design basis, an adequate level of confidence would be attained if it were known that 70 percent of the structures placed in similar environments had shown deterioration at a particular number of years of service. Therefore, by substituting 70 percent into Eq. 1,

$$R_a = \frac{424}{K^{0.42}} \tag{8}$$

in which R_a is the number of years when 70 percent of the structures constructed of average concrete and placed in an environment of K (chlorides in parts per million) would have visual evidence of corrosion of the reinforcing steel in the substructures.

The time for deterioration of a reinforced concrete substructure would probably be proportional to the rate of moisture evaporation of the concrete to the considered depth for various design variables. Therefore, an equation encompassing differences in the evaporative characteristics of an average and a specific concrete is

$$R_t = R_a \times \frac{Q_a}{Q_s} \tag{9}$$

in which

R_t = years to deterioration of bridge substructure;
R_a = years to deterioration of an average bridge substructure;
Q_a = quantity of water evaporated by average concrete; and
Q_s = quantity of water evaporated by specific concrete.

Substitution of Eqs. 7 and 8 in Eq. 9 gives,

$$R_t = \frac{10^{0.0442C}\, C^{0.717}\, S_i^{1.22}\, 1,011}{K^{0.42}\, W_m^{1.17}} \tag{10}$$

Figure 9 shows a solution of Eq. 10. Figure 10 shows how this equation calculated from the data for average construction practice fits the field conditions as observed for the inspected bridges in California.

Correlations to Independent Tests

There is a general lack of detailed data in the literature that would independently test the accuracy of the results of this investigation. One of the more detailed investigations that have been reported in sufficient scope is that by Lea and Watkins (9). Their test of the protective value of concrete in preventing the corrosion of reinforcing steel was started in 1929 at two locations in England. One exposure site was in natural sea water at Sheerness, and the other was at Watford in sea water concentrated to three times its normal salinity.

At each of the test sites, concrete piles of 5 in. square in section and 5 ft long were exposed to the two environments. The piles were constructed of mixes that varied from ½ to 2 in. of slump; 3.8, 6.4, and 10.6 sacks of cement per cubic yard; and 1 and 2 in. of cover over the reinforcing steel. Unfortunately, this test was not continued under the original test conditions beyond 10 years. However, comprehensive data are available for the first 10 years of controlled exposure and are compared to the estimated time to deterioration given in this study (Table 1).

In an attempt to correlate the empirical equation of California experience to the reported test data, a statistical comparison was made by the method of least squares for the piles exposed at Sheerness. The authors stated that the exposure conditions at Watford were not as severe as Sheerness; therefore, the statistical comparison was not made for this location. For six degrees of freedom, the level of significance was approximately 0.01 with a correlation coefficient of 0.809. The standard error of estimate was $\pm$ 1.92 years.

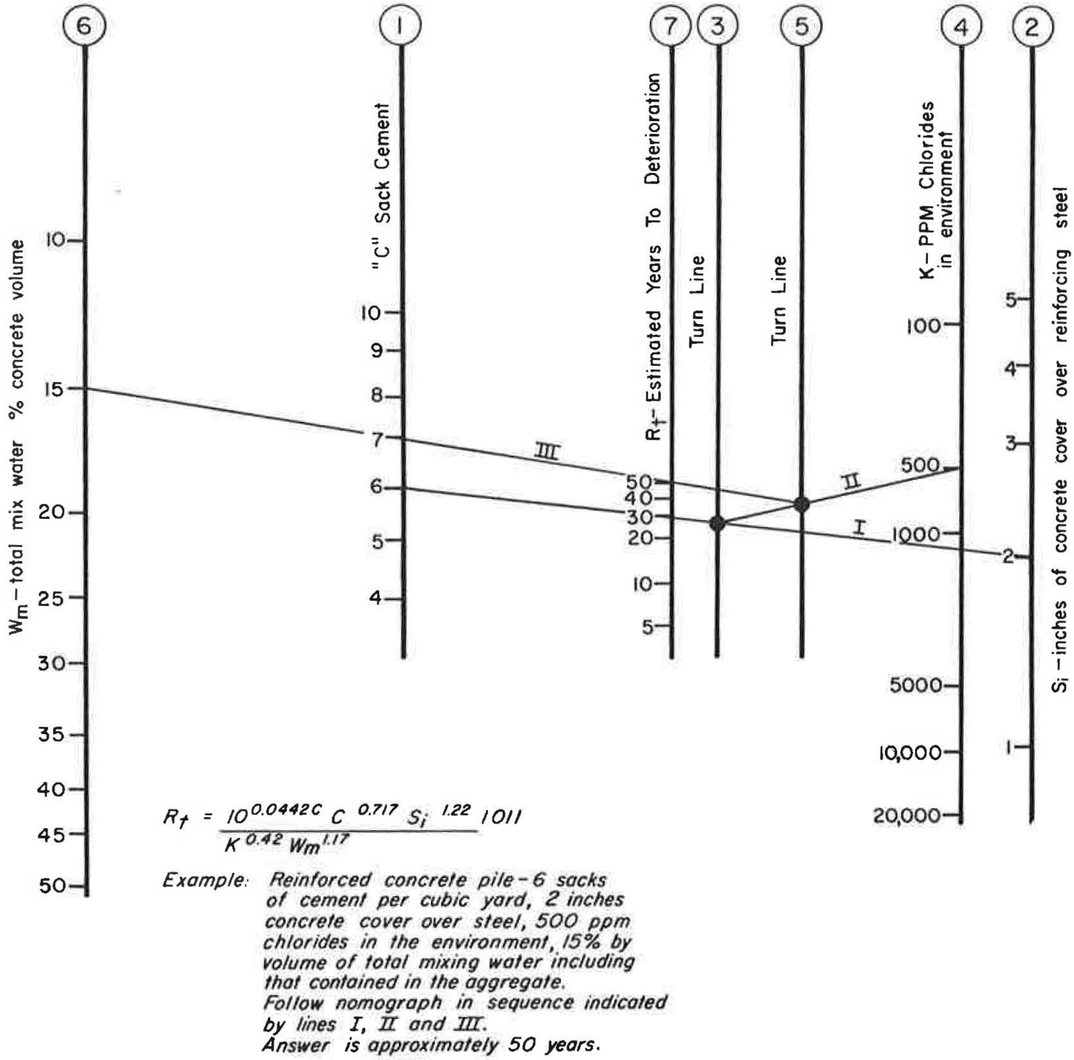


Figure 9. Chart for estimating deterioration time for reinforced concrete pile.

Due to the lack of details as to when the reinforced concrete test piles had cracked in the long-time study of cement performance (10), it is not possible to correlate those test results directly to the estimated performance results of this study. Even so, it was stated in the 10-year report "that for the salt water exposures the effect of rusting and expansion of the reinforcing steel is a major deteriorating influence." All the concrete piles were so constructed that there was 1.5 in. of concrete cover over the reinforcing steel. The concrete tested consisted of the following mixes: Mix 1, 5 sacks of cement per cu yd and 2 in. slump; Mix 2, 7 sacks, 2-in. slump; Mix 3, 7 sacks, 8-in. slump. The estimated time to corrosion of the reinforcing steel for the piles made of the reported concrete mixes was less than 8 years. At the St. Augustine test site, the data showed that after 15 years exposure, for Mix 1, 89 percent of the total piles were cracked; for Mix 3, 47 percent were cracked, and 16 percent of the Mix 2 piles were cracked. After about 15 years of exposure at the Corona Del Mar test site, 100 percent of the Mix 1 piles were cracked, whereas 86 percent of the Mix 3 and none of the Mix 2 piles were cracked.

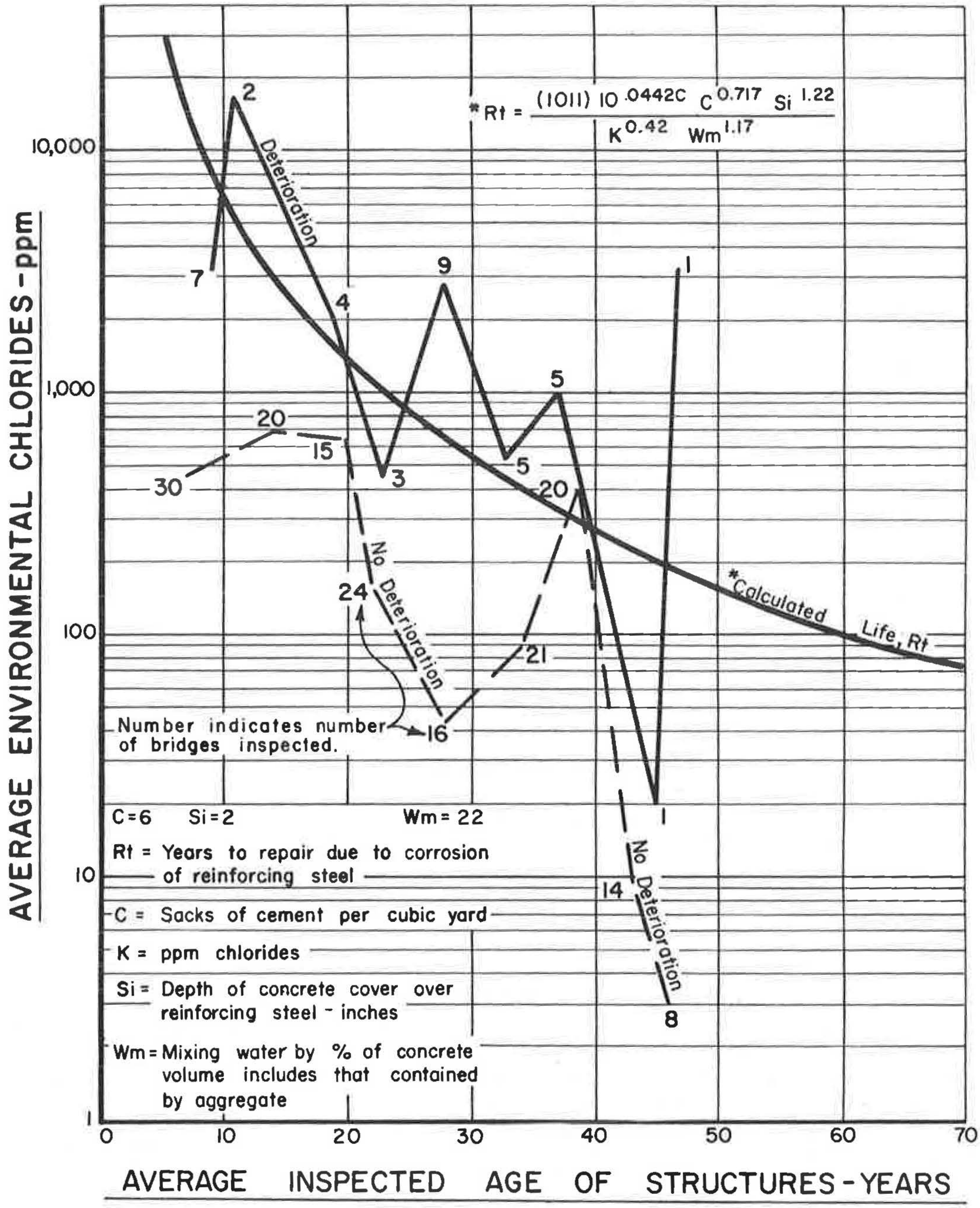


Figure 10. Condition of 205 bridges vs average age at inspection and chloride concentration.

TABLE 1

COMPARISON OF THEORETICAL AND ACTUAL TEST DATA[a] FOR REINFORCED CONCRETE PILES

(Sacks/Cement cu yd)	Concrete (in.)		Age at Cracking[b] (yr)			
			Sheerness		Watford	
	Slump	Cover	Actual	Estimated[c]	Actual	Estimated[c]
10.6	½	1	6	8	-[d]	5
		2	-[e]	20	-[d]	11
	2	1	10	8	-[e]	4
		2	-[e]	18	-[d]	10
6.4	½	1	3	4	1	3
		2	10	10	-[e]	6
	2	1	2	4	1	2
		2	10	9	-[e]	5
3.8	2	1	3	2	3	1
		2	8	5	6	3

[a]Data obtained from (9) as results of field exposure tests in England.
[b]Of 5-in. by 5-in. by 5-ft test pile.
[c]Based on Eq. 10.
[d]Test conditions altered after 10 years, cracked in 10 to 20 years.
[e]Test conditions altered after 10 years, not cracked in 20 years.

Although the actual time when a pile had cracked due to reinforcing steel corrosion was not reported, the deterioration of the piles apparently occurred at an early age, which confirms the corrosion time computations to a limited degree. However, the estimated time to deterioration as indicated by this study does not infer a degree or rate of deterioration at any location. The data indicate the time to the incidence of deterioration.

As a result of corrosion of the steel in reinforced concrete bridge piles in Texas, extensive repair was required after approximately 7.5 years of service (12). The original concrete mixture in this structure consisted of 5 sacks of cement per cu yd, 6.5 gal of water per sack of cement, and 2 in. of cover over the reinforcing steel.

The stream water was reported to be "loaded" with chlorides. The total quantity of chlorides was not reported, although deposits of salts were clearly visible on the surface of the banks of the stream. Assuming a concentration of chlorides equal to that found in sea water, which could be considered as "loaded" quantity, the estimated years of service to corrosion of a pile in this structure would be 8 years. Based on the assumption of the quantity of chlorides in the Texas bridge environment, the field data agree with the estimated life.

The results of an inspection of reinforced concrete structures on the east coast of the United States reported in 1957 (6) showed that 6 out of 10 structures had reinforcement corrosion at an average inspected age of 23 years. Although neither construction details nor the initial time when the structures were found to be deteriorated was reported, the inspection indicates that the normal construction methods, which were used until 1946, were not adequate to insure an indefinite maintenance-free service life of structures exposed to sea water. In sea water, the expected time to corrosion of piles based on a 6-sack mix, with 15 percent total water by volume and 2 in. of cover is 10 years.

ANALYSIS

From the viewpoint of an investigator, it has been extremely fortunate that the basic design of reinforced concrete substructures in California has not varied within drastic limits for the past 50 years. Otherwise, it would have been exceedingly difficult, or even impossible to use the data given in this paper to correlate the condition of the inspected structures with the environment. Even though a correlation was found, the data only reflect the normal exposure of bridge substructures for that portion which can be observed. Underwater or underground sections were not investigated, and the application of these data to those conditions should be tempered with performance data.

For the purpose of comparing variables in concrete design, a direct ratio (of the evaporative characteristics) was established between the average concrete used for 50 years and the various types exposed to the laboratory test conditions. It is speculated that this relationship of relative concrete evaporative characteristics would be consistent with that observed in other environments. On this basis, care should be used in applying these data to structures not directly exposed to the free flow of air, such as that found in highway bridges. For instance, 70 percent of the highway bridges built with the recommendations of concrete design by some California harbor departments (13) and consequently exposed to sea water would be expected to have corrosion of reinforcing steel after approximately 13 years of service. Evidently, there must be an environmental difference in the exposure conditions between a highway bridge and harbor facilities on the California coast, and between various harbor departments. For instance, Gaye and Agatz (14), commenting on the construction of dock works in Germany during the 1920's, indicated that 10 cm of covering is necessary to protect the iron elements from corrosion. Using their described concrete mixtures, a California bridge would probably show deterioration of the substructure in approximately 20 years. However, these German structures were built of concrete to which trass had been added to the mix. English tests (9) have shown that, for comparable conditions, concrete made of trass cements had proved better than portland cement for inhibiting the corrosion of reinforcement. Also, results of the tests performed in the United States indicate that the rate of deterioration of reinforced concrete piles varies not only with the basic mixes used but also with the type of cement (10).

There appear to be many factors (6, 8, 15) that could influence the deterioration of a structure exposed to a corrosive environment, such as the influence of the type of cement (6, 8, 9, 10, 13); variables of aggregate size, grading, or manufacture (13); additives (9, 14, 15, 16, 17, 18); workmanship (2, 6, 8, 13, 14, 18); curing (6); and the environment (15, 19, 20). These factors require considerable attention when designing a structure for a specific maintenance-free service life.

SUMMARY

A total of 239 reinforced concrete bridges were inspected throughout the State of California, and it was found that corrosion was attacking the embedded steel in bridges that were in widely dispersed geographic locations. A study of the data indicated that the primary cause of corrosion was the presence of chlorides in the environment. Apparently, the chlorides permeate the concrete, and, after a period of exposure, depending on chloride concentration in the natural soils or waters, bring about the corrosion of the reinforcing steel.

Based on mathematical distribution curves, it was found that 70 percent of the structures placed in an environment of a certain chloride concentration would have corrosion of the reinforcement according to Eq. 8. By the method of least squares, it was found that this formula duplicated the distribution curve of the field results within about 7 percent of the actual percentage of structures affected by corrosion.

The past history of bridge construction in the State of California indicates that the average reinforced concrete piles have generally been made of a mixture of 6 sacks of cement per cubic yard of concrete and 2 in. of concrete covering over the steel.

A mathematical analysis of the data indicates that the concentration of sulfates in the environment is not the primary cause of reinforcing steel corrosion. It is speculated that the presence of excessive sulfates could lead to the corrosion of reinforcing after it had caused chemical attack and possibly disintegration of the concrete.

Laboratory tests were made of the moisture absorption and evaporative characteristics of various concrete mixes consisting of combinations of the following: 4, 6, 8 sacks per cubic yard; 2, 4, and 6 in. of slump; aggregate of 1.5, 4, and 10 percent absorption by weight; and various mixtures of neat cement bars. When these test specimens were exposed in a fog room at 73.4 F and 100 percent relative humidity, the calculated rate moisture absorption of the various mixes could not be distinguished from each set of specimens. Evidently, the variables of the fog environment were such that the small differences in the absorption rates of the concrete mixes could not be detected.

When the concrete specimens were exposed to the dry room at 73.4 F and 50 percent relative humidity, it was found that the rate of moisture evaporation at the calculated depth would vary according to Eq. 7.

Because of the relationship of the chloride concentration in the environment to the deterioration time of the structures, it was assumed that the time to the build-up of a harmful quantity of salts was related to the volume of water that could evaporate in the various mixes and thus would deposit an accumulating quantity of chlorides in the concrete. Eq. 7, which gives the relationship of the various mixes to the rate of moisture evaporation, was used with Eq. 9 to give Eq. 10, the basic equation for estimating the probable number of years to corrosion of the reinforcing steel in 70 percent of the structures placed in the normal highway bridge environment.

This derived equation (Eq. 10) does not imply that a structure will be in structural distress at the indicated time. It only represents the number of years that could be expected to elapse before corrosion of the embedded steel could cause rust stains or cracking of the concrete. The equation does not indicate the rate at which corrosion will occur in different substructure members of the same bridge, only when the first member (pile) will be visibly affected. It is expected that the longer the period to the first evidence of corrosion, the greater the time to the visible distress of the last member of the same structure.

Correlating these data to those reported by others was difficult because of a lack of construction or environmental details. However, in the exposure tests of piles in England, the indicated age to distress as determined by this test method was within approximately 2 years of the actual time required for distress for the 10 years of controlled environmental conditions. Other comparisons were made between the reported conditions of structures elsewhere in the United States, and although the test could not be related in terms of years, it did indicate whether a designer could anticipate the premature deterioration of the bridge substructure.

The work presented in this paper should not be thought of as the "final answer." Instead, it is hoped that a step has been made toward finding a means to evaluate the effect of the environment on the durability of reinforced concrete bridge piles, and that attention has been directed to the kinds of data needed to predict service life.

ACKNOWLEDGMENTS

These investigations were conducted as a part of the activities of the Materials and Research Department of the California Division of Highways. The authors wish to express their appreciation to F. N. Hveem, Materials and Research Engineer, and to Bailey Tremper, Supervising Materials and Research Engineer, retired, for their advice and direction during these studies. They also wish to thank the numerous personnel of the California Division of Highways and Bridge Department who extended their aid and cooperation during this study.

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