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16. ABSTRACT

In the past 50 years of highway construction in the State of California, hundreds of reinforced concrete bridges have been constructed. The majority of these structures have required a minimum of maintenance attributable to the corrosion of reinforcing steel. However, there are a number of bridges giving evidence of cracking due to steel corrosion.

In California the most serious example of costly maintenance has been the repair of the deterioration 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 upon the existing technology and a concrete coverage of one or two inches over the reinforcing steel.

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

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STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

ENVIRONMENTAL INFLUENCE ON THE CORROSION OF REINFORCING STEEL IN CONCRETE BRIDGE SUBSTRUCTURES

By

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and

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ington, D.C.
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environment. A mathematical relationship was derived that describes the percentage of structures that had deterioration for any concentration of chlorides in the immediate area.

Laboratory tests continuing over a period of 2½ years indicated a relationship between the cement and water contents of concrete 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 that indicates the probable time to cracking of a reinforced concrete substructure. The developed equation is based upon 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.

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INTRODUCTION

In the past 50 years of highway construction in the State of California, hundreds of reinforced concrete bridges have been constructed. The majority of these structures have required a minimum of maintenance attributable to the corrosion of reinforcing steel. However, there are a number of bridges giving evidence of cracking due to steel corrosion.

In California the most serious example of costly maintenance has been the repair of the deterioration 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 upon the existing technology and a concrete coverage of one or two inches over the reinforcing steel.

In addition to the 1917 inspection of the condition of American-made structures, an inspection by the Committee of the Institution of Civil Engineers on structures in British waters in 1920 also revealed the deterioration of numerous reinforced concrete structures[3]. More recent reports[4,5,6] have shown that there continues to be distress of reinforced concrete structures in marine environments. On the basis of these reports of deterioration of coastal structures, and the experience in California, it

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appears that cost of maintaining these structures due to corrosion of steel 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 that was constructed in the arid Colorado Desert of California. It was found that this structure had the same evidence of distress as was found on 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 of 1950-51, and 20 of the 66 bridges or about 30 percent of the structures were found to have corroding reinforcing steel. From the data collected during this investigation, it appeared that the high concentration of salts in the soil (flow is usually not present at these bridge sites except once or twice yearly) was responsible for the observed distress of the structures. Those bridges which were placed in soils of high salt concentrations (up to 41,000 parts per million of chlorides) showed distress while 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. The following are the results of this investigation.

Condition of Bridges

The environments in which bridges in California were inspected for possible corrosion of reinforcing steel were:

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(1) coastal, (2) valley, (3) desert, and (4) mountain. The 239 bridges that were inspected varied in age from up to 50 years. The condition 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 to be indicative of the corrosion of reinforcing steel. Typical examples of these observed conditions are shown in Photographs Nos. 1 and 2. Photo 1 depicts the cracking of reinforced concrete piles in a 10 year old bridge exposed to tidal water near San Diego, California. Photo 2 shows an area of advanced concrete spalling and the exposed reinforcing steel on the San Francisco-Oakland Bay Bridge.

Of the 239 bridges that were 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 to be outside the scope of this specific investigation. Based upon this criterion, there were 37 structures which had 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 which had visual evidence of deterioration in members other than the substructure were not used in the analysis that follows. Also, 5 structures that 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

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a possible correlation to the condition of the structures: (1) pH or hydrogen-ion concentration, (2) sulfates as SO_4, (3) chlorides, as Cl, that were contained in the soils and, if any, the natural drainage waters, and (4) the electrical resistance of the soils and drainage waters.

Chlorides and Concrete Distress

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 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 versus deterioration and also versus the environmental factors.

Figure 1, Influence of Chlorides on the Deterioration of Reinforced Concrete Bridge Piles or Walls, shows the method that was used for plotting the "raw" data. On this chart it will be noted that 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 chloride concentration of the environment. Figure 1 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 which had visual evidence of deterioration in other than substructure members, 5 non-deteriorated 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 subsequent to the initial investigation; these preceeding data were not included as a part of the original data reduction. However, with judgment some of the preceeding data were included in the chart shown as Figure 8.

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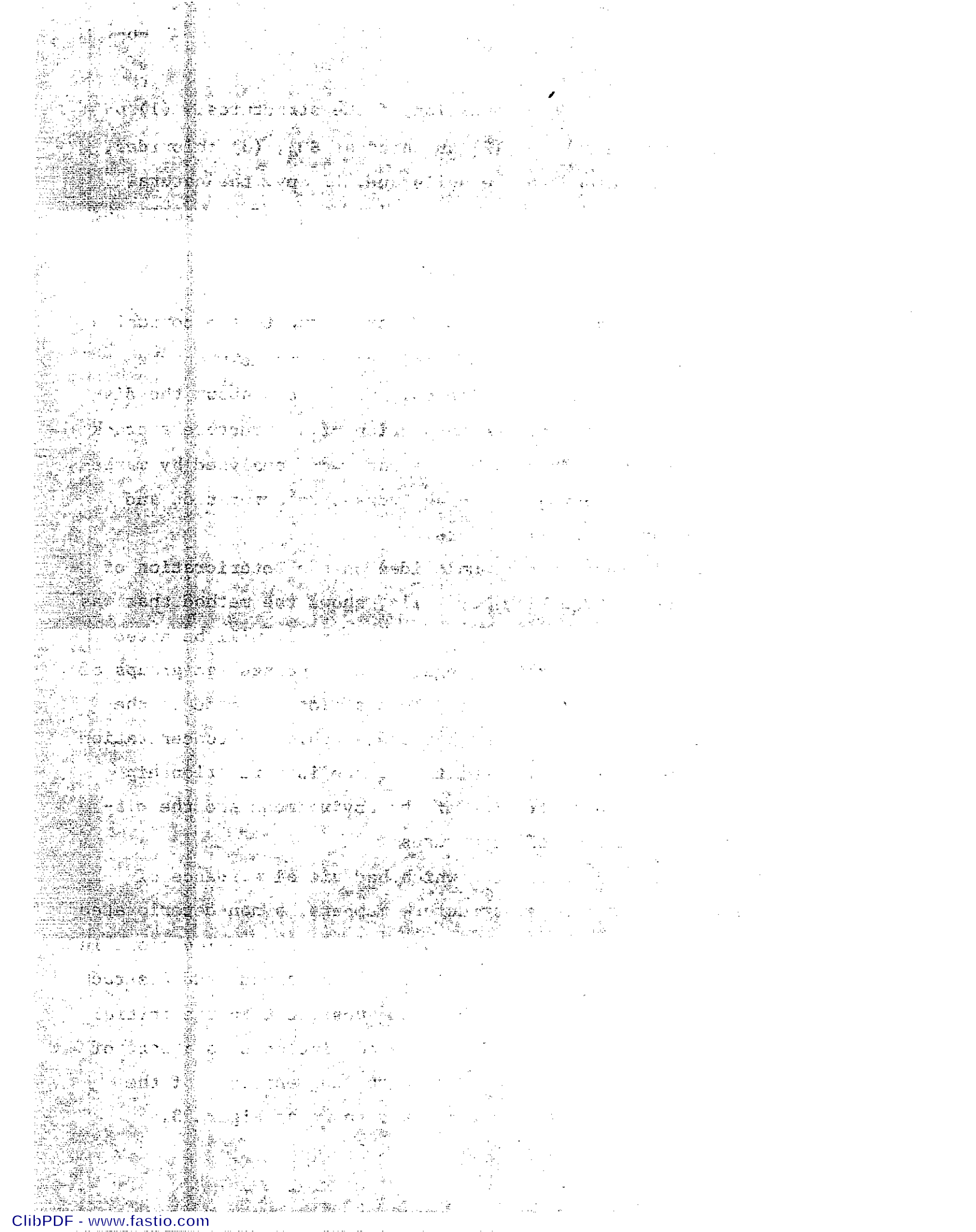

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The data represented by Figure 1 were then tabulated into distribution curves of the percentage of structures deteriorated at each of the four time increments. The data were then mathematically analyzed to determine the yearly rate at which the structures would deteriorate in each chloride concentration. The result of the analysis is shown by Figure 2.

The correlation equation was found to be: Equation 1: $R_y = 0.165\ K^{0.42}$, wherein R_y = percent of total structures showing distress per year, and K is the chloride concentration in parts per million found in either the soil or water that is in physical contact with the substructure. After the equation was derived, it was tested for its ability to duplicate the original distribution curve of the condition of the bridges.

By the method of least squares, a statistical correlation indicated 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.

It should be clearly understood that these data represented by Equation 1 are representative of a distribution curve of the observed condition of a number of bridges. Analyzing the performance of a single structure would require a thorough and detailed 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 the difference in the environment, workmanship, curing, etc. For example, these data are for sections of a bridge substructure that were visible to the investigator. The incidence of deterioration below ground or under water is not considered to be applicable to analysis by these data as it was not studied.

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The records indicate that the concrete normally used for the construction of bridge piles consisted of six sacks of cement per cubic yard of concrete, and the design provided two inches of concrete cover over the reinforcing steel.

Sulfates and Concrete Distress

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

The result of 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, generally there was a high chloride content also.

This observation suggested that there could be an interdependence between the chloride and sulfate concentration and the deterioration rate of the structures.

In order 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 outlined 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 are shown by Figure 3, Influence of Sulfate and Chloride Concentration on Bridge Deterioration, and indicate that the more imminent cause of reinforcing steel corrosion is the chlorides present in the environment.

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Study of Concrete Variables

A. Mixes

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

It is considered that the corrosion of steel in concrete is the result of 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 (1) in a fog room at 73.4° F. and 100% R.H. and (2) to drying at 73.4° F and 50% R.H. (for periods of about four months). It was observed that the 3"x3"x11.25" concrete blocks would approach moisture equilibrium after three to four months of exposure.

In all cases the concrete specimens were made in triplicate with the following variables: (1) aggregate having 1.5%, 4%, and 10% absorption by weight, (2) equivalent of 4, 6, and 8 sacks of Type II cement per cubic yard, and (3) slump of 2, 4, and 6 inches. The maximum aggregate size was 100% passing the 3/4" screen. In addition, neat cement bars of 1"x1"x11.25" 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.

All of 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 2.5

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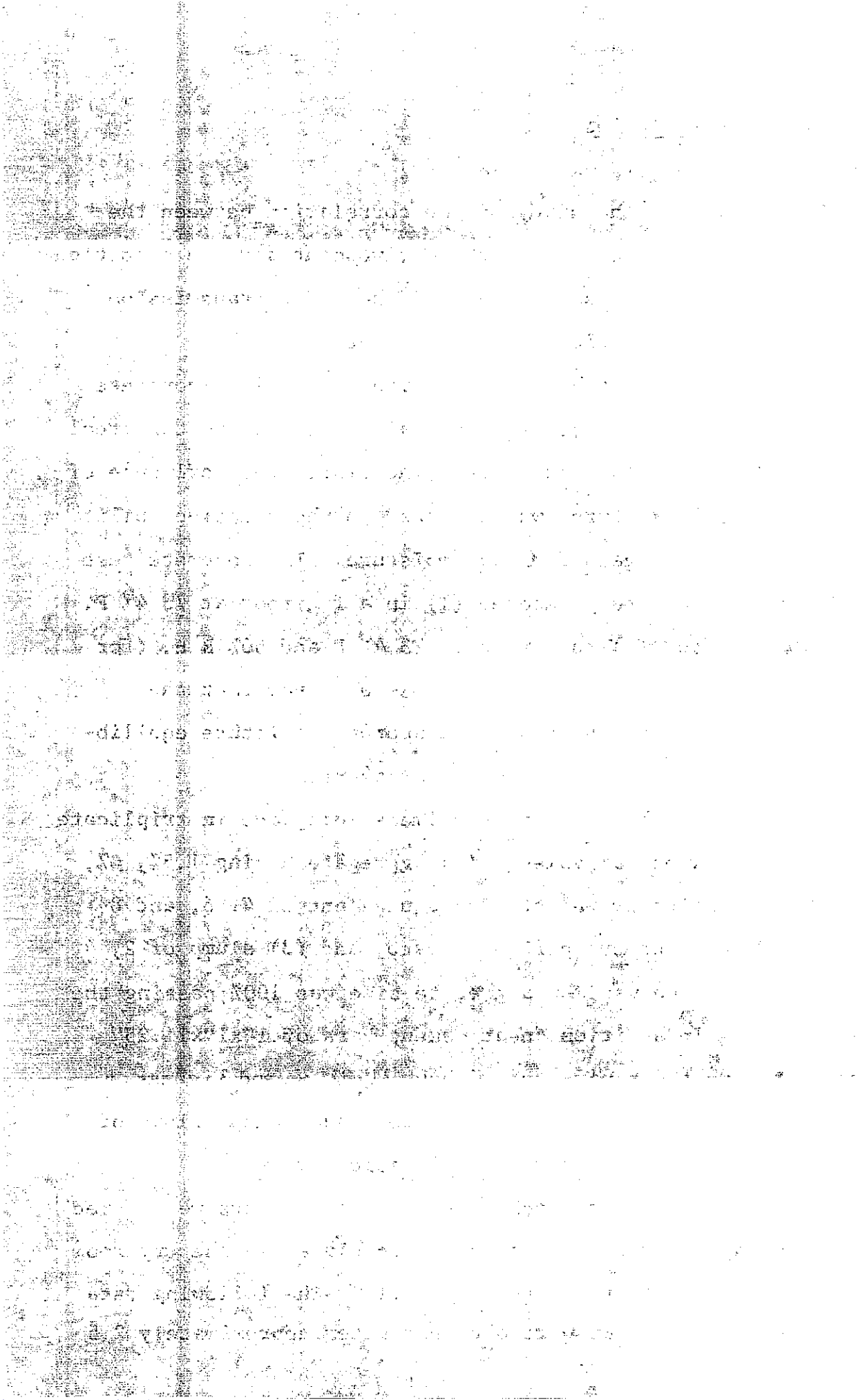

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years of alternately exposing the concrete to the fog and drying room atmospheres. The study of the concrete blocks is continuing.

Water Voids

There are various methods utilized 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, it was desired that the measurement of the gain and loss of moisture be on a continuing basis, so as 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 utilized to describe the volume of water that is absorbed and evaporated by concrete that is subjected to the alternate conditions of 50% relative humidity at 73.4° F and 100% R.H. at 73.4°F. None of the concrete specimens was subjected to other than the described environmental conditions except for the rare occasions when it was necessary to mechanically adjust the environmental rooms.

The weight gain and loss of moisture of the concrete specimens were 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 in excess of 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

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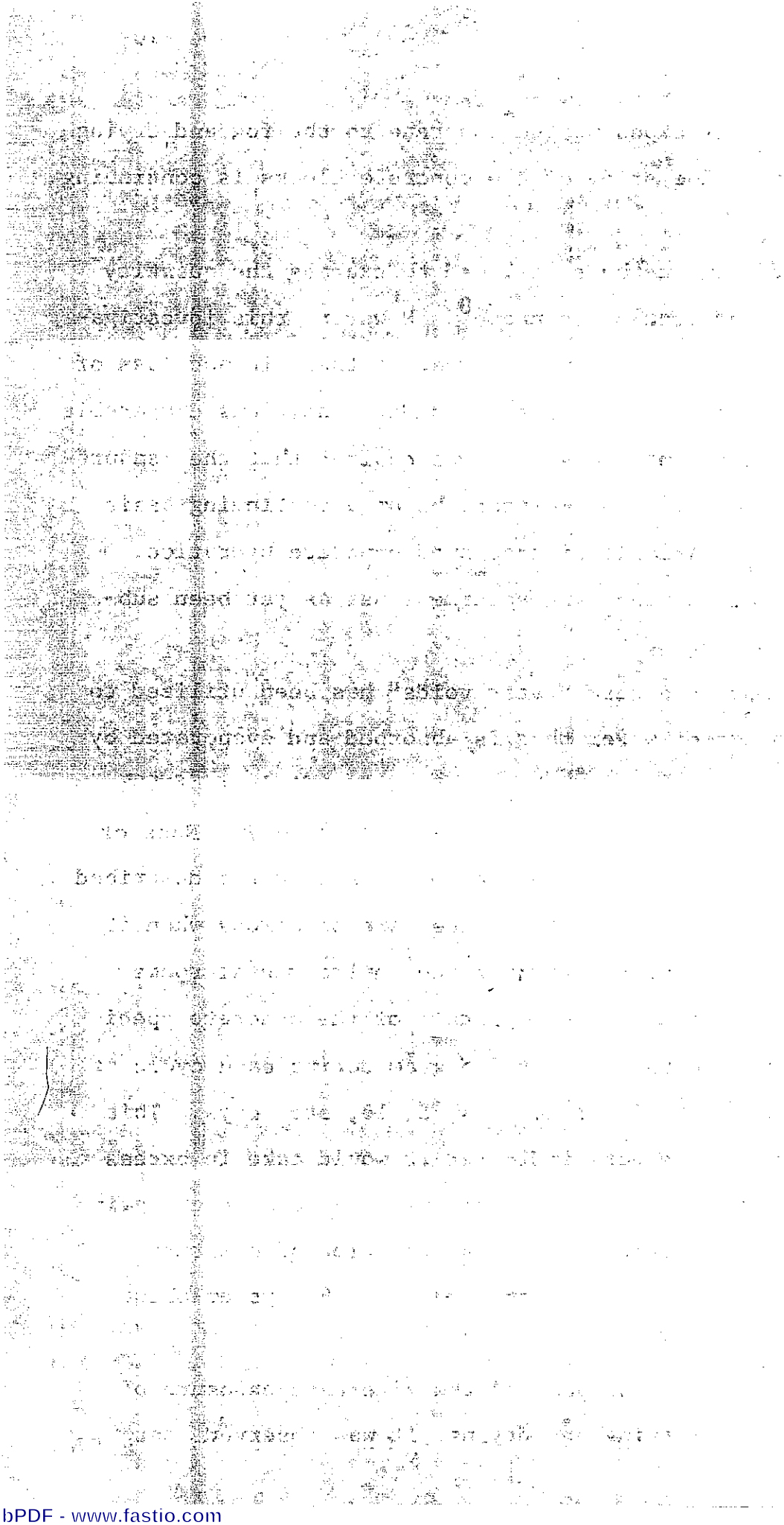

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the weight of water gained and lost for each triplicate set of concrete blocks was approaching a reproducible value. Therefore, the measurements of moisture movement in this study that are considered to be significant are for a period of exposure of the specimens which begin after the concrete has aged approximately one year. Concrete that is greater than one year old is termed "mature" concrete.

It has been previously mentioned that there were three types of aggregate utilized in the test with absorptions of 1.5%, 4%, and 10% by weight. The use of this range of aggregates resulted in concrete that varied in weight from about 150 to 110 pounds per cubic foot. 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 which could occur as a result of extremes in concrete densities.

Figure 4, Water Voids Versus Mixing Water in Mature Concrete, is a plot of the relationship found 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 indicates that the relative volume of water which was 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

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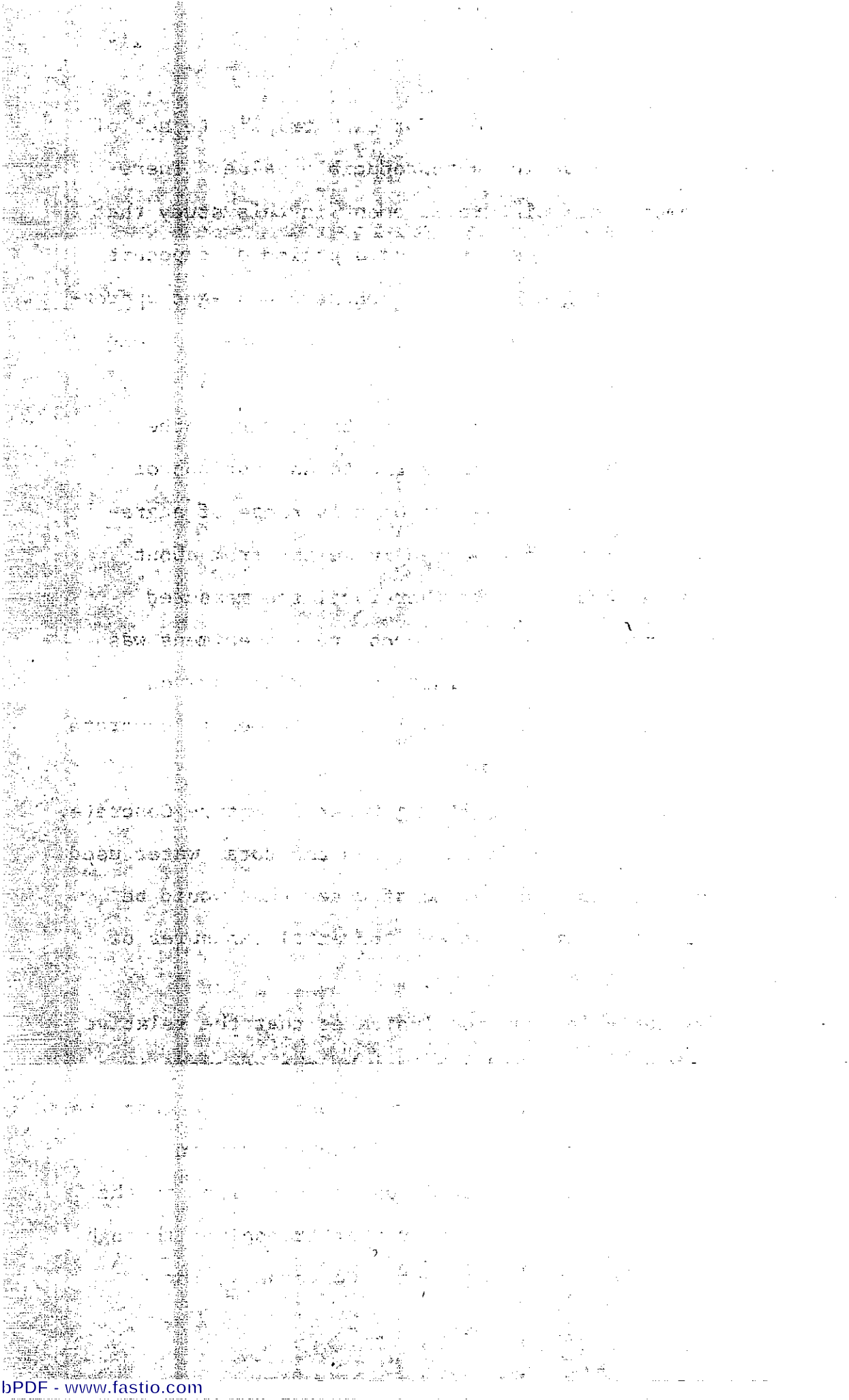

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given total mixing water content, an increase in cement content reduced the quantity of water that was absorbed or evaporated by the concrete under these test conditions.

The equation which described the measured gain and loss of moisture in mature concrete is:

Equation 2. $$V_w = \frac{W}{V} \times 100 = \frac{0.85\ (W_m)^{1.17}}{C^{0.717}}$$

V_w = water voids in percent of concrete volume as measured by loss in drying at 50% R.H. and gain by exposure in the fog room

W = volume of water gained and lost

V = volume of concrete

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

C = sacks of cement per cubic yard of concrete

Variations in Water Voids

During the first year of measuring the water voids, or volume of moisture that transpired through the concrete test specimens, it was observed as indicated on Figures 5 and 6 that this quantity of moisture movement would not be reasonably duplicated by each subsequent cycle of exposure. It was observed that under the test conditions it was approximately 200 days after the concrete specimens were cast that a maximum total movement of water would occur. This maximum movement of water appeared to occur when the blocks were placed in their first exposure to the fog room. The concrete made with the aggregate having 10% absorption was the exception.

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

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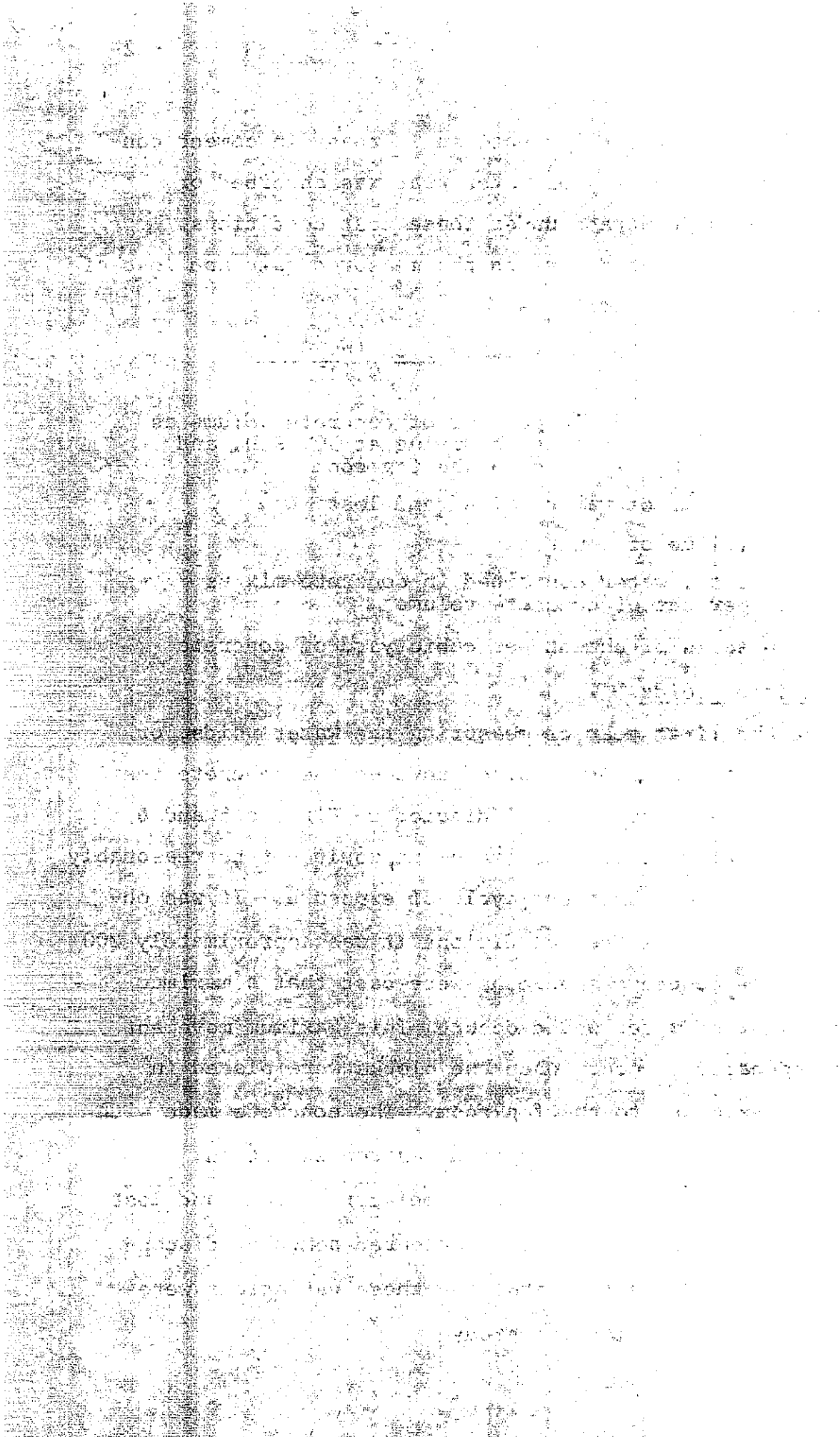

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Figure 5, Influence of Cement Content on the Volume Change of Water Voids -- Aggregate Constant, indicates that the cement content and time are the variables that are responsible for the early differences in the volume of moisture absorption and loss 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 which were made with the equivalent of 8 sacks of cement per cubic yard reduced about 35% from their maximum volume of moisture movement while 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 6, Influence of Aggregate on Volume Change of Water Voids - Cement Contents Constant.

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

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 not found to be constant but decreased with the time of exposure. This observation was not considered to be unusual when one considers that 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

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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 because this was measured 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 on the basis of relative volume. Therefore, when a concrete block was in a saturated condition, and it 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. Since 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 utilized 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 while another may be saturated. The reality of this assumption is highly speculative, as would be other assumptions on the relative degree of contained moisture content at various points within the concrete mass.

An analysis of the data indicated that there were two predominant variables that influenced the calculated rate of loss

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of moisture during the exposure in the dry room at 73.4° F and 50% R.H. These variables were: (1) the cement content and (2) the assumed volume of the concrete that contained the evaporable water. A plot of these data is described by the following relationship:

Equation 3: $t_y = 10^{0.0442 C_{S_i}^{2.22}} 0.086$

t_y = time in years to dry to equilibrium with an environment of 73.4° F and 50% R.H.

C = sacks of cement per cubic yard of concrete

S_i = depth below surface in inches

The 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% R.H. inferred that the concrete mix variables were not primary test variables. For a calculated depth of approximately 75% of the half depth of the concrete, all specimens absorbed moisture at a rate of about 0.2 inches 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. It should be pointed out that although the blocks were exposed in a fog room of controlled temperature and humidity, the fog dispersion within the room is not uniform. For this reason the concrete blocks exposed to the fog room 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

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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.

Relating Laboratory and Field Studies

It has been determined by field exposure tests that the protection of reinforcing steel provided by concrete is a variable which depends upon 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, (1) the absorption and (2) the rate of moisture movement through the concrete.

Because of the test methods utilized in this study, there is no direct measure of the relationship of the saturated surface dry to oven dry method for measuring concrete absorption of water. However, it is surmised that the quantity of absorbed water found in this study could be related to the amount determined in oven drying method.

The concrete variables observed in this study were related to the performance of the concrete in an atmospheric type of environment. It is believed that the behavior of concrete in an atmospheric type of environment is similar to the performance of concrete exposed near the ground or water line. This

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assumption is based upon the measured differentials in the chloride content of concrete that has been exposed to a marine environment[1]. In this previously reported study, it was shown 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 described by the equation $Q = T_y W_v$, where 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, Equation 3 indicates the time in years when concrete would dry to equilibrium to a calculated depth and is: $T_y = 10^{0.0442 C S_i 2.22} 0.086$ where T_y = Time in years to dry at 73.4° F and 50% R.H., C = Sacks of cement per cubic yard of concrete, and S_i = the calculated depth of drying in inches below the surface.

To obtain the rate or cycles of drying per year, it is necessary to take the reciprocal of the expression, thus the rate per year of drying is:

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

The volume of water as a percent of concrete volume that would be evaporated from mature concrete is described by Equation 2:

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

V_w = water voids in percent of concrete volume

W = volume of water gained and lost, 73.4° F at 50% R.H. and 73.4° F at 100% R.H.

V = volume of concrete = AS_i = Area, A, times Depth, S_i, in inches

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

C = sacks of cement per cubic yard of concrete

By combining Equations 2 and the reciprocal of Equation 3, it is possible to obtain an expression that relates the evaporative characteristics of the concrete in these test conditions, which is:

$$Q_y = T_y V_w V$$

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

V = volume of concrete, cubic inches

If Equations 2 and 3 are combined, Equation 4 will be derived and indicates the comparative yearly volume of water which could be evaporated by concrete. Equation 4 is:

$$Q_y = \frac{0.0988(W_m)^{1.17} A}{10^{0.0442C} C^{0.717} S_i^{1.22}}$$ = cubic inches of water evaporated per year at 50% R.H., 73.4° F, (A = cross-sectional area in sq.in.)

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% by volume, total mixing water, and S_i = 2" of concrete cover over the reinforcing steel. Using these basic figures, the yearly quantity of water evaporated from an average concrete pile which is saturated with water and exposed to 73.4° F and 50% R.H. could be: using Equation 4, $Q_a = \frac{0.0988(W_m)^{1.17} A}{10^{0.0442C} C^{0.717} S_i^{1.22}} = 0.2356A$

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Equation 1 described the percentage per year of the total structures that were found in the field to have corrosion of the reinforcing steel. On a design basis, it is considered that an adequate level of confidence would be attained if it were known that 70% of the structures which are placed in similar environments had shown deterioration at a particular number of years of service. Therefore, by substituting 70% into Equation 1, Equation 5 would be:

$Ra = \frac{424}{K^{0.42}}$ Ra = the number of years when 70% 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.

On a speculative basis, it is suggested that the time for deterioration of a reinforced concrete substructure would be proportional to the rate of moisture evaporation of the concrete to the considered depth for various considered design variables. Therefore, an equation which can be formulated to encompass differences in the evaporative characteristics of an average and a specific concrete can be described as follows:

$R_t = R_a \times \frac{Q_a}{Q_s}$ wherein R_t = years to deterioration of bridge substructure

R_a = years to deterioration of an average bridge substructure

Q_s = quantity of water evaporated by specific concrete

By substitution of Equations 4 and 5 and reducing:

Equation 6: $$R_t = \frac{10^{0.0442 C_c^{0.717} S_i^{1.22} 1011}}{K^{0.42} W_m^{1.17}}$$

A graphic solution of this formula is shown on Figure 7, Chart for Estimating Deterioration Time for Reinforced Concrete Pile. Figure 8, Condition of 205 Bridges Plotted Against Average Age at Inspection and Chloride Concentration, shows how Equation 6 calculated from the data for average construction practice fits the field conditions as observed for the inspected bridges in California.

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

TABLE I

Sacks of Cement per cu. yd.	Concrete		Sheerness--Years[2]		Watford--Years[2]	
	Slump	Cover	Actual	Estimated[3]	Actual	Estimated[3]
10.6	½"	1"	6	8	*	5
		2"	**	20	*	11
	2"	1"	10	8	**	4
		2"	**	18	*	10
6.4	½"	1"	3	4	1	3
		2"	10	10	**	6
	2"	1"	2	4	1	2
		2"	10	9	**	5
3.8	2"	1"	3	2	3	1
		2"	8	5	6	3

1 Test data obtained from Reference 9 and are the results of field exposure tests in England.

2 Years to cracking of test pile - 5" x 5" x 5'.

3 Estimate is based upon the Equation $R_t \doteq \dfrac{10^{0.0442} C_C^{0.717} S_i^{1.22} 1011}{K^{0.42} W_m^{1.17}}$

* Test conditions altered after 10 years, cracked in 10 to 20 years.

** Test conditions altered after 10 years, not cracked in 20 years.

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Correlations to Independent Tests

Generally there is a 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 which have been reported in sufficient scope is that by Lea and Watkins[9].

Their reported 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 inches square in section and 5 feet long were exposed to the two environments. The piles were constructed of mixes that varied from ½" to 2" of slump, 3.8, 6.4, and 10.6 sacks of cement per cubic yard and 1" and 2" of cover over the reinforcing steel. Unfortunately, this test was not continued beyond 10 years under the original test conditions. However, comprehensive data are available for the first 10 years of controlled exposure and are compared to the estimated time to deterioration as based upon this study. This comparison is shown on Table I.

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 so the statistical comparison was not made for this location. For 6 degrees of freedom, the level of significance was approximately 0.01 with a correlation coefficient of 0.809. The standard error of estimate was found to be $\pm$ 1.92 years.

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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 directly correlate their test results to the estimated performance as indicated by the 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 of the concrete piles were constructed so that there was 1.5 inch 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" of slump. Mix 2, 7 sacks, 2" slump. Mix 3, 7 sacks, 8" slump. The estimated time to corrosion of the reinforcing steel for the piles made of the reported concrete mixes is less than 8 years. At the St. Augustine test site, their data showed that for Mix 1 (5 sack, 2" slump) 89% of the total piles were cracked, Mix 3 (7 sacks, 8" slump) 47% were cracked, and 16% of the Mix 2 (7 sacks, 2" slump) piles were cracked after 15 years. At the Corona Del Mar test site, 100% of the Mix 1 (5 sacks, 2" slump) were cracked, 86% of the Mix 3 (7 sacks, 8" slump), and none of the Mix 2 (7 sacks, 2" slump) piles were cracked after about 15 years of exposure.

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 and confirms the corrosion time computations to a limited degree. It should be pointed out that 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.

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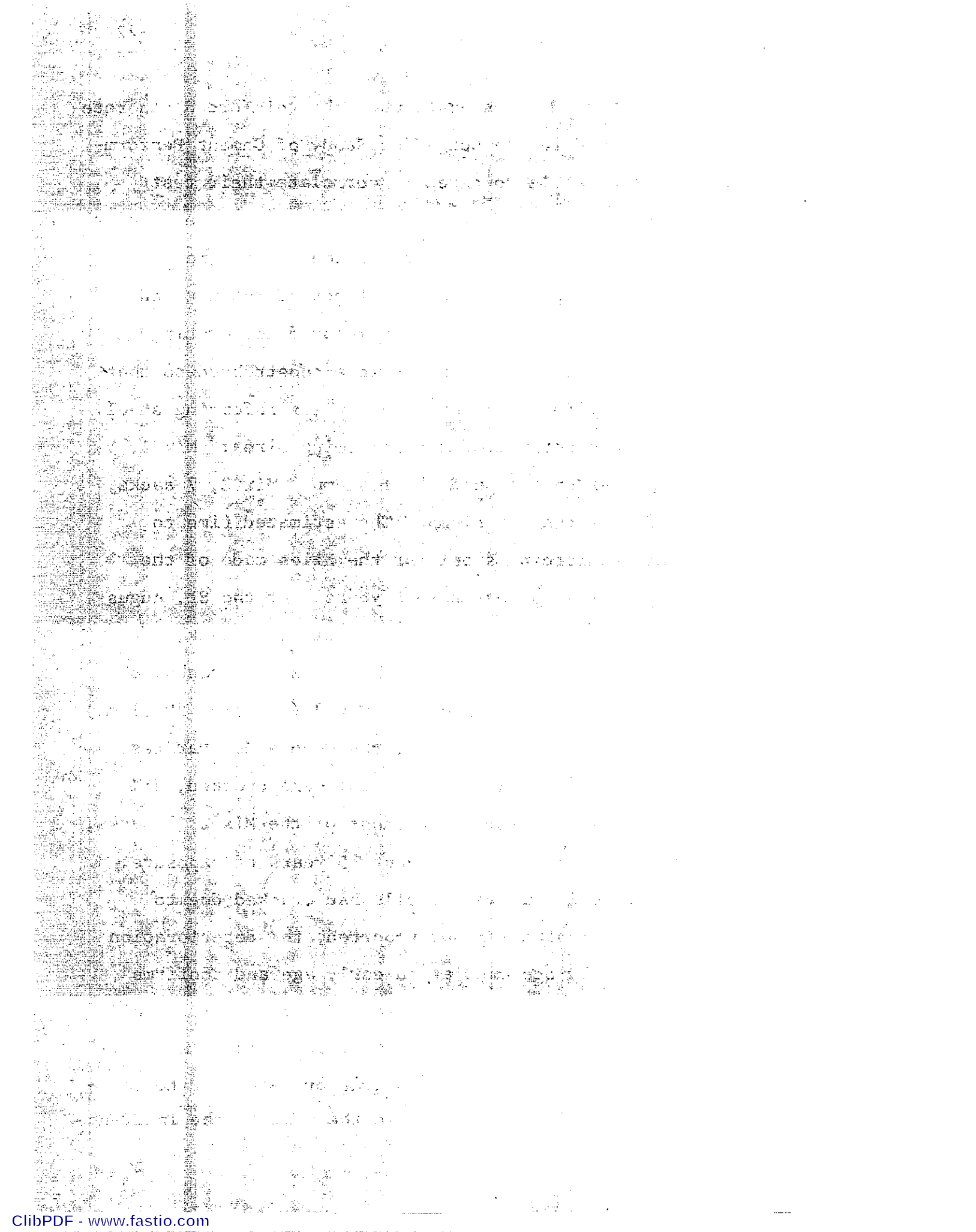

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As a result of corrosion of the steel in reinforced concrete bridge piles in Texas, they required extensive repair after approximately 7.5 years of service[12]. The original concrete mixture in this structure consisted of 5 sacks of cement per cu. yard, 6.5 gallons of water per sack of cement and 2 inches 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 upon 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 U. S. A. were reported in 1957[6] and 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 were reported, the inspection indicates that the normal construction methods, which were utilized 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 upon a 6 sack mix, with 15% total water by volume and 2 inches of cover is computed to be 10 years.

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DISCUSSION

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, use of the data shown in this paper to correlate the condition of these inspected structures with the environment would have been exceedingly difficult or impossible. Even though this correlation was found, it should be remembered that the data 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, it appears that careful consideration should be given to the use of these data when applied to structures that are not in direct exposure to the free-flow of air such as that found in highway bridges. For instance, 70% of the highway bridges built and exposed to sea water with the recommendations of concrete design by some California Harbor Departments[13] would be expected to have corrosion of reinforcing steel in 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

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between various harbor departments. For instance, Gaye and Agatz[14], commenting on the construction of dock works in Germany during the 1920s, indicated that 10 centimeters of covering is necessary to protect the iron elements from corrosion. Using their described concrete mixtures, a California bridge probably would show deterioration of the substructure in approximately 20 years. It should be pointed out that these German structures were built of concrete to which trass had been added to the mix. The English tests[9] have shown that for equal conditions, concrete made of trass cements had proved to be better than a Portland cement in 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 utilized but also with the type of cement[10].

There appear to be numerous 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.

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SUMMARY

Two hundred thirty-nine reinforced concrete bridges were inspected throughout the State of California, and it was found that corrosion was attacking the embedded steel 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 upon chloride concentration in the natural soils or waters, resulting in the corrosion of the reinforcing steel.

Based upon mathematical distribution curves, it was found that 70% of the structures which were placed in an environment of a certain chloride concentration would have corrosion of the reinforcement according to the following relationship:

$$R = \frac{424}{K^{0.42}}$$

R is the observed years of service when 70% of the structures had corrosion of the reinforcing steel, and K equals parts per million chlorides in the soil or water. By the method of least squares, it was found that this formula duplicated the distribution curve of the field results within about 7% of the actual percentage of structures that were 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" of concrete covering over the steel.

A mathematical analysis of the data indicates that the concentration of sulfates (SO_4) in the environment is not the primary cause of reinforcing steel corrosion. It is speculated that the

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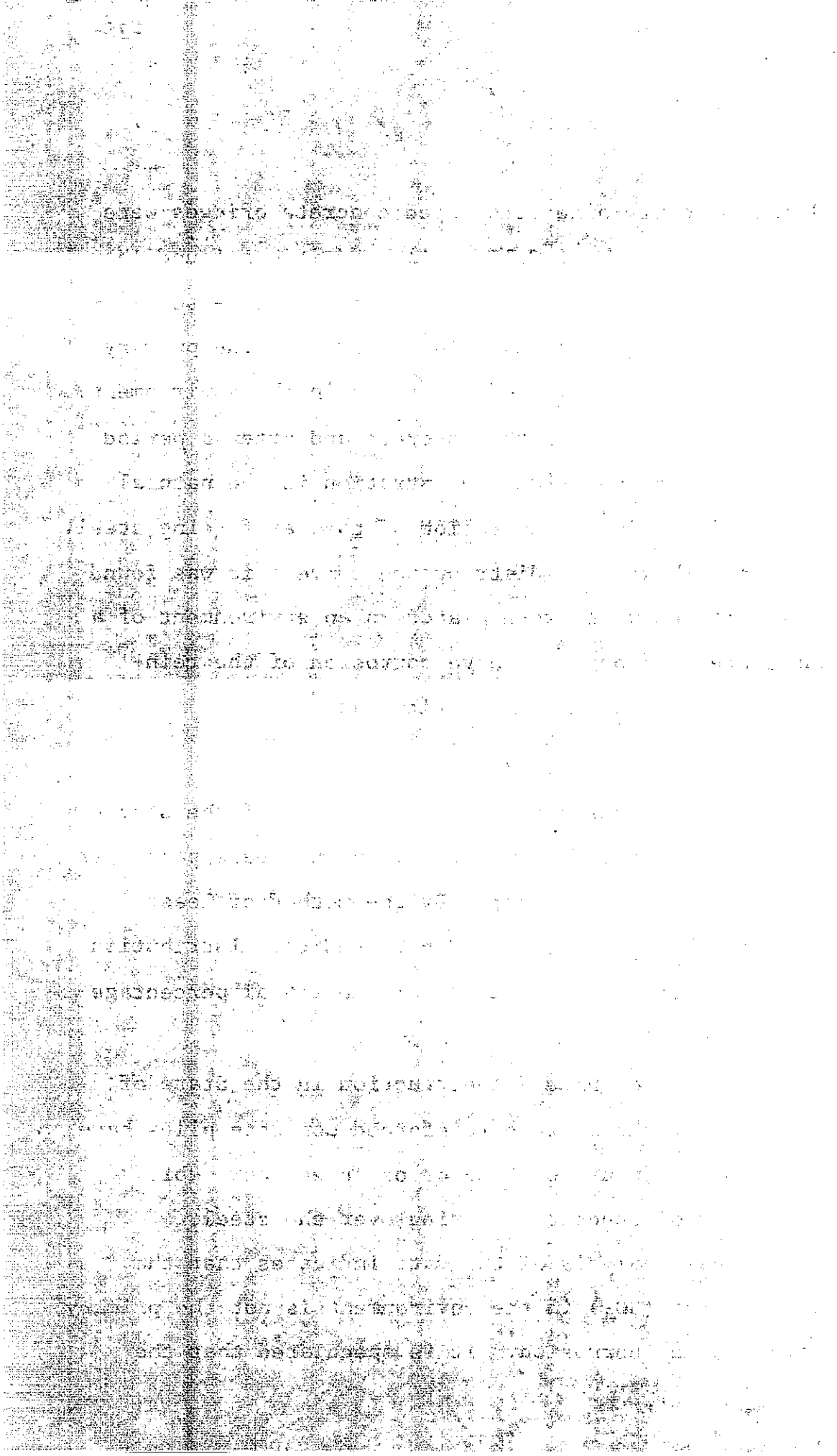

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presence of excessive sulfates could lead to the corrosion of reinforcing after it had caused chemical attack and possibly disintegration of the concrete.

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 yard; 2, 4, and 6 inches of slump; aggregate of 1.5, 4 and 10 percent absorption by weight; and also various mixtures of neat cement bars. When these test specimens were exposed in a fog room at 73.4° F and 100% R.H., it was found that 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% R.H., it was found that the rate of moisture evaporation at the calculated depth would vary as the following relationship derived from the test data:

$$Q = \frac{0.0988(W_m)^{1.17}A}{10^{0.0442C}C^{0.717}S_i^{1.22}}$$

Q = cubic inches of water evaporated per year in 50% R.H. and 73.4° F

W_m = total mixing water as percent of concrete volume

C = sacks of cement per cubic yard of concrete

S_i = depth below surface in inches

A = area of concrete in square inches

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

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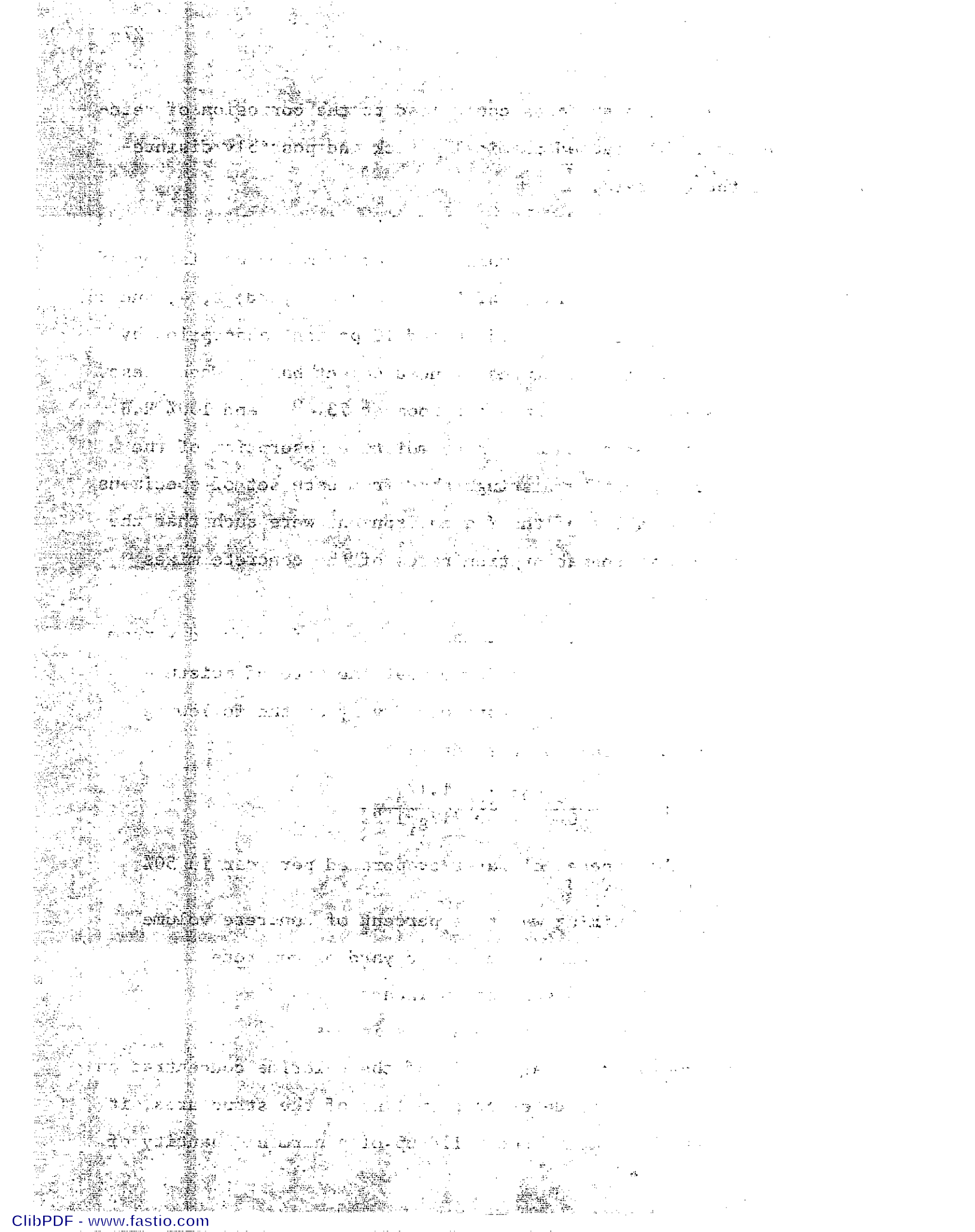

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salts would relate to the volume of water that could evaporate in the various mixes, and thus deposit an accumulating quantity of chlorides in the concrete. The previous equation, which showed the relationship of the various mixes to the rate of moisture evaporation, was utilized in a ratio of the evaporative characteristics of the average concrete to a specific concrete in the following manner:

$R_t = R \frac{Q \text{ average}}{Q \text{ specific}}$ which becomes the basic equation for estimating the probable number of years to corrosion of the reinforcing steel in 70% of the structures placed in the normal highway bridge environment, which is:

$$R_t = \frac{10^{0.0442} C_C^{0.717} S_i^{1.22} 1011}{K^{0.42} W_m^{1.17}}$$

It should be pointed out that this derived equation does not imply that a structure will be in structural distress at the indicated time. It only represents the number of years which could be expected to elapse before corrosion of the embedded steel could cause rust stains and/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.

In an attempt to correlate this data to that reported by others, it was found that a correlation was difficult because of a lack of construction or environmental details. However, in the exposure tests of piles performed in England, the indicated age to distress as determined by this test method is within approximately 2 years of the actual time required for distress for the 10 years

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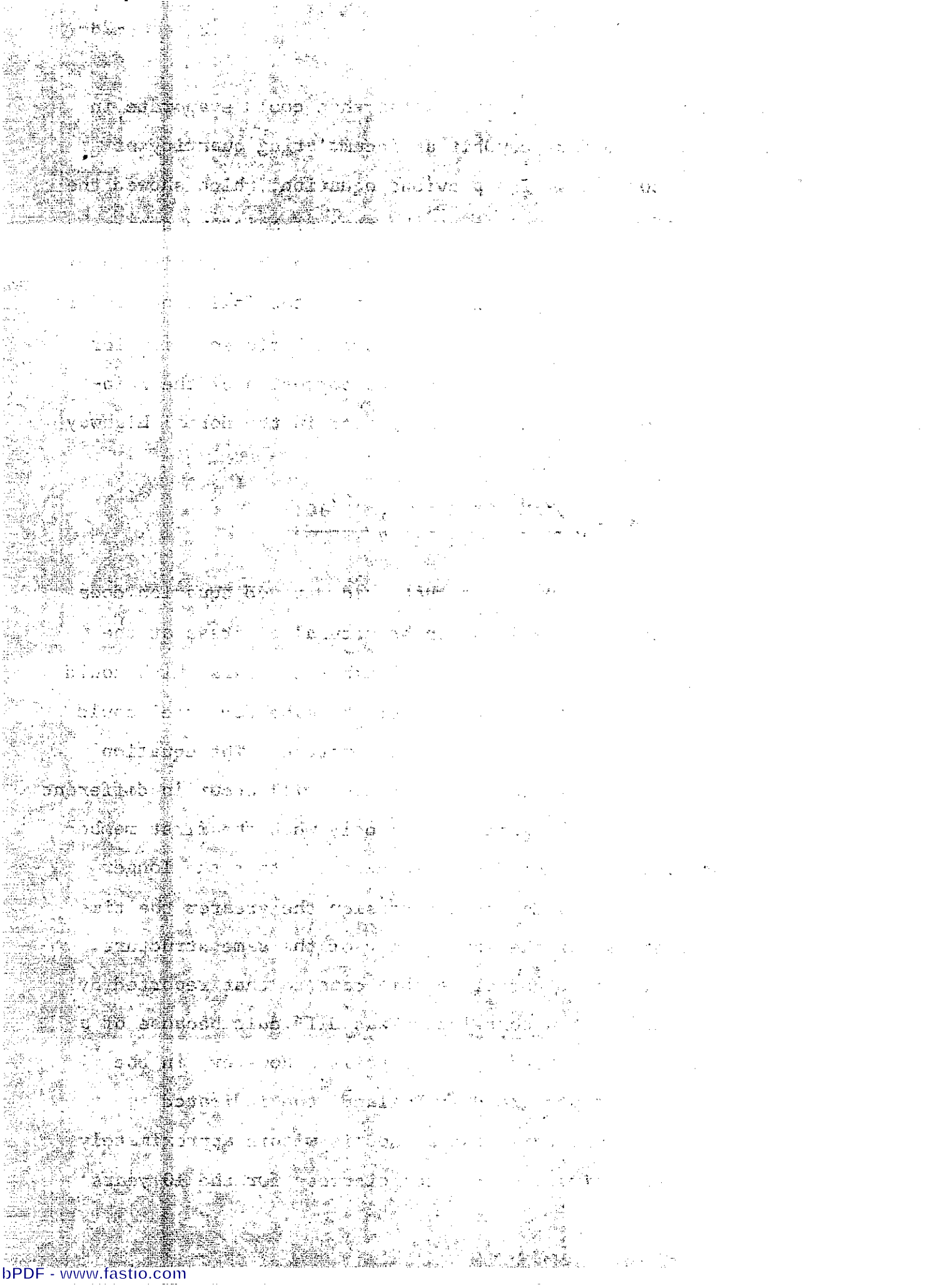

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of controlled environmental conditions. Other comparisons are made between the reported conditions of structures elsewhere in the United States, and although the test could not be related in terms of years, it did indicate whether a designer could 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 we have made a step toward finding a means to evaluate the affect of the environment on the durability of reinforced concrete bridge piles, and that we have directed attention to the data needed to predict service life.

ACKNOWLEDGMENTS

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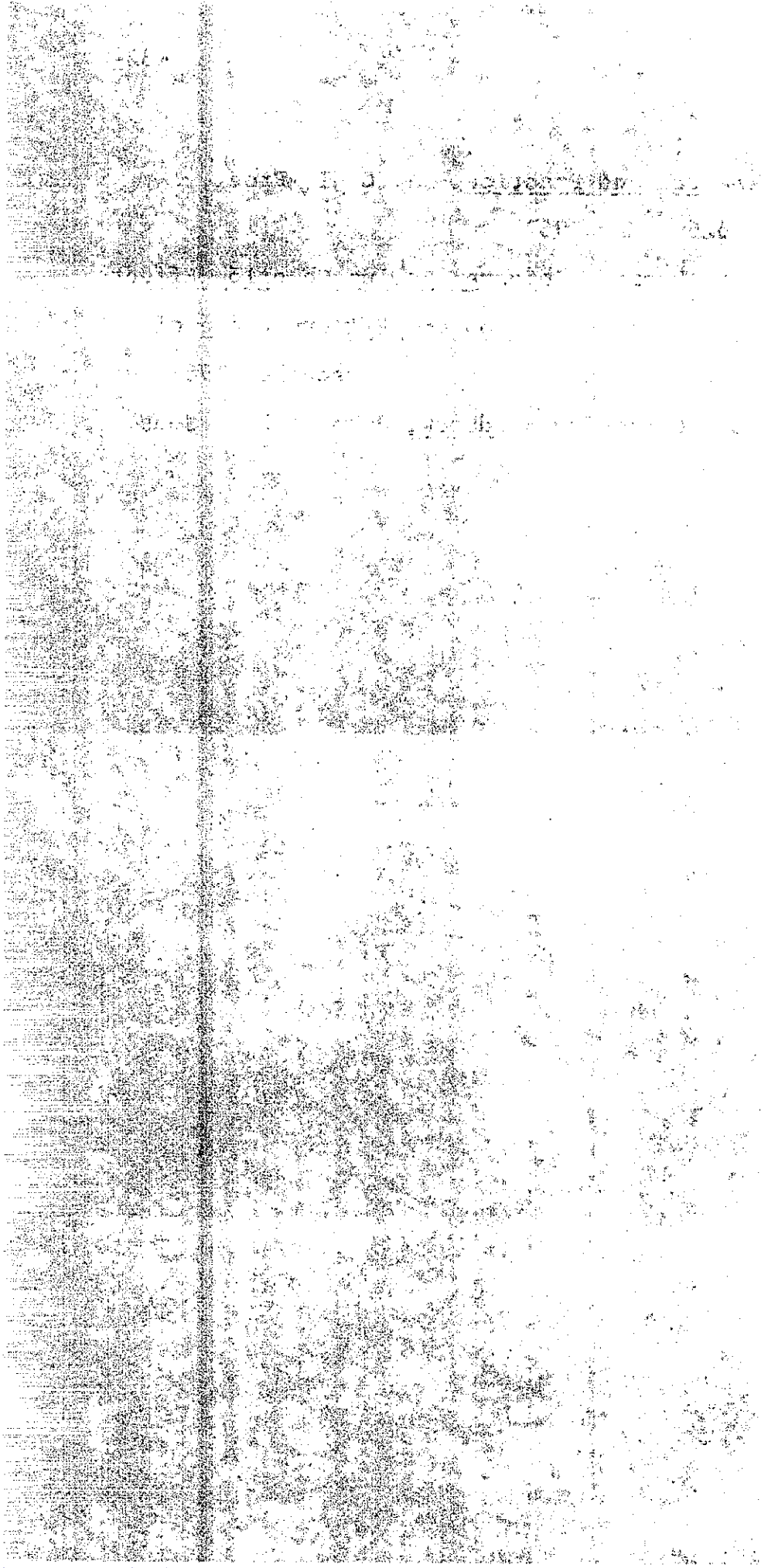

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FIGURE 1

INFLUENCE OF CHLORIDES ON THE DETERIORATION OF REINFORCED CONCRETE BRIDGE PILES OR WALLS

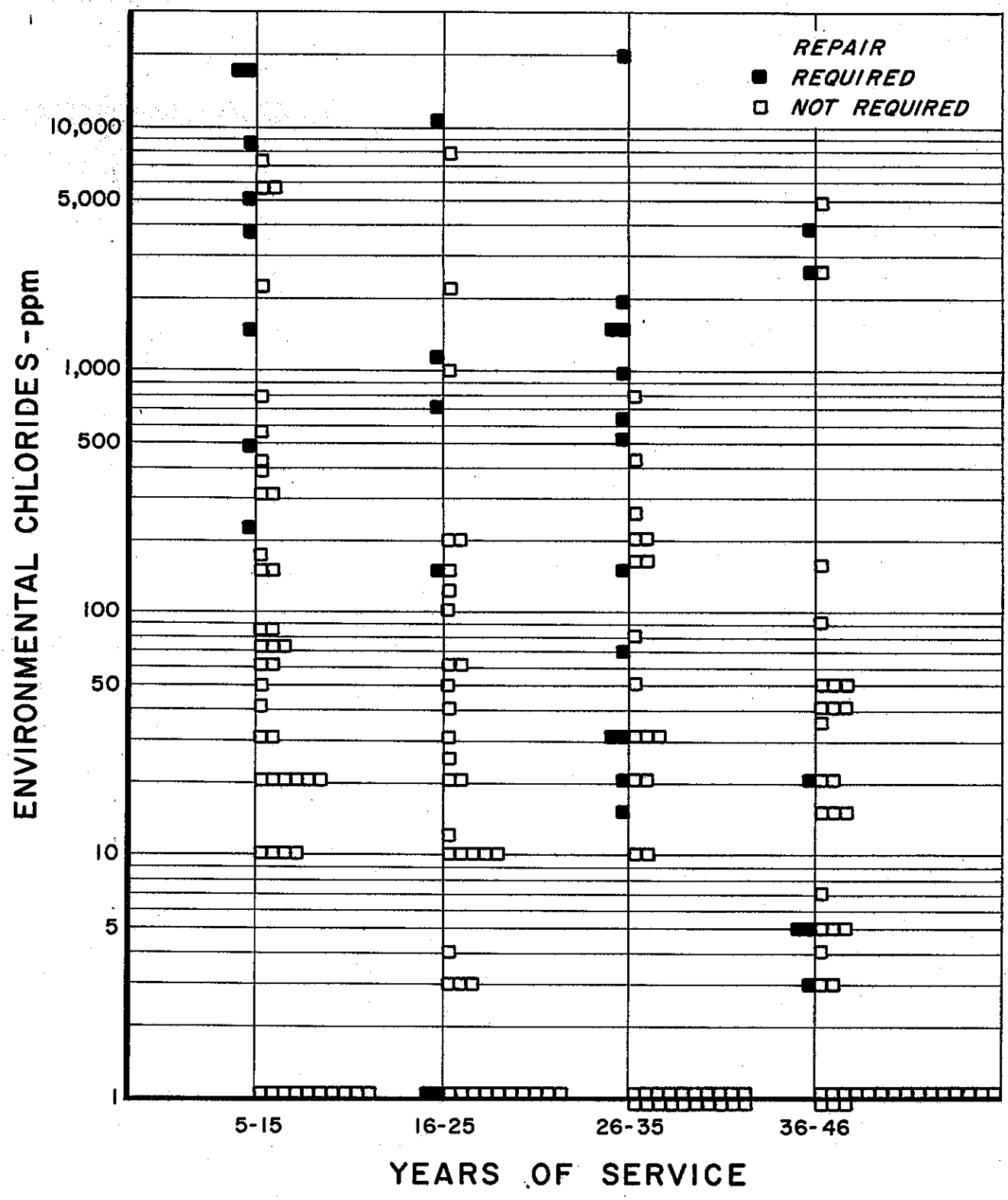

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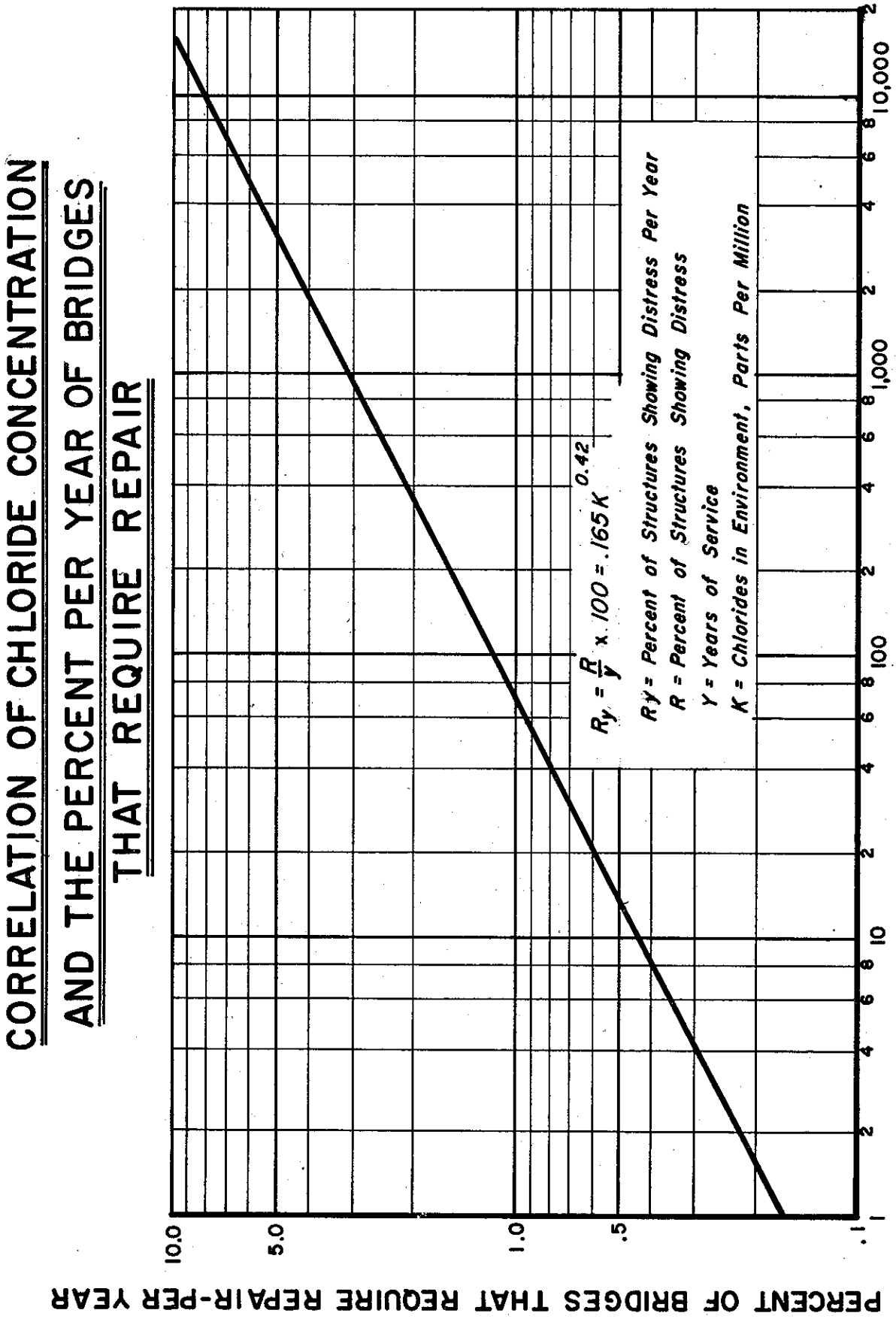
Figure 2



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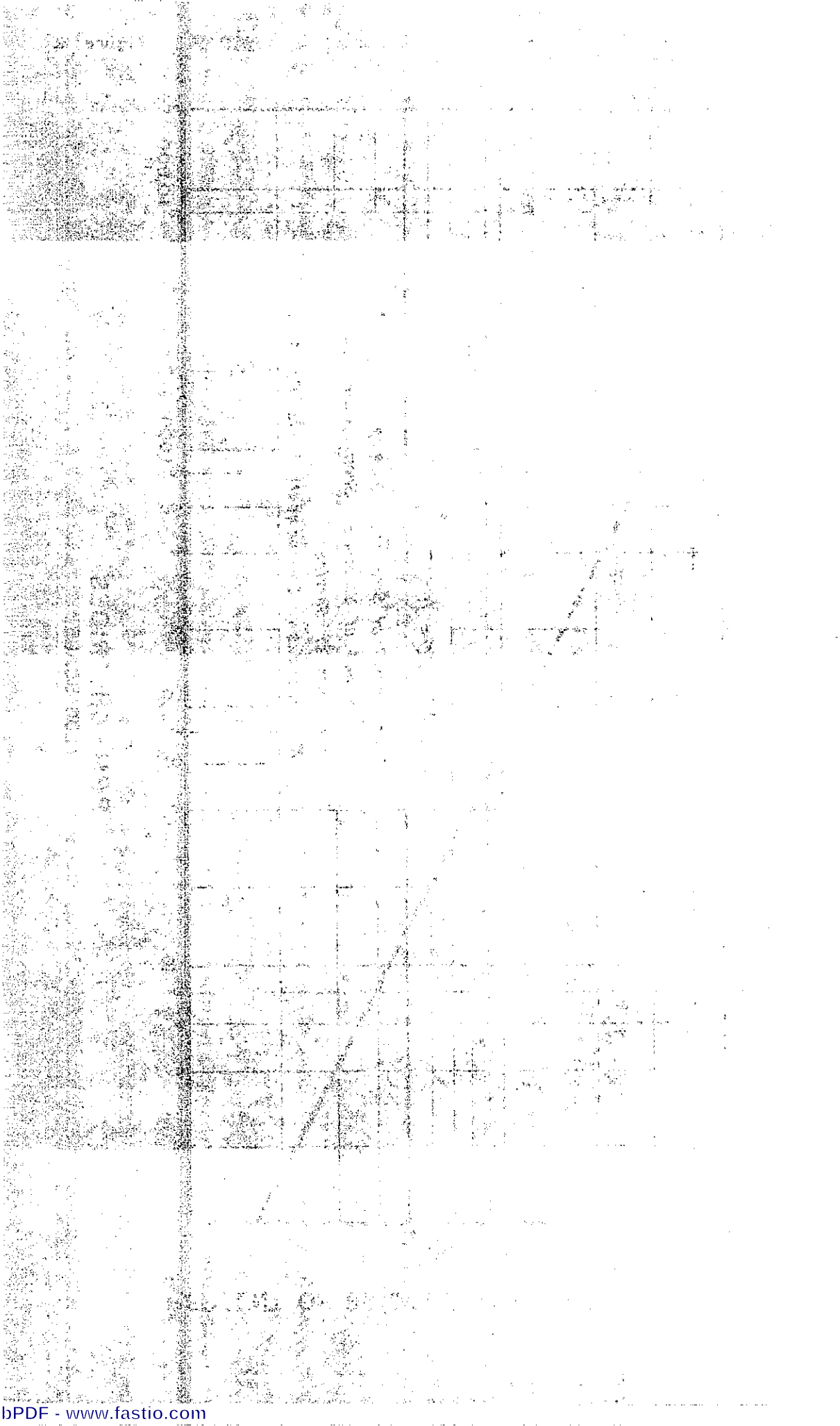

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FIGURE 3

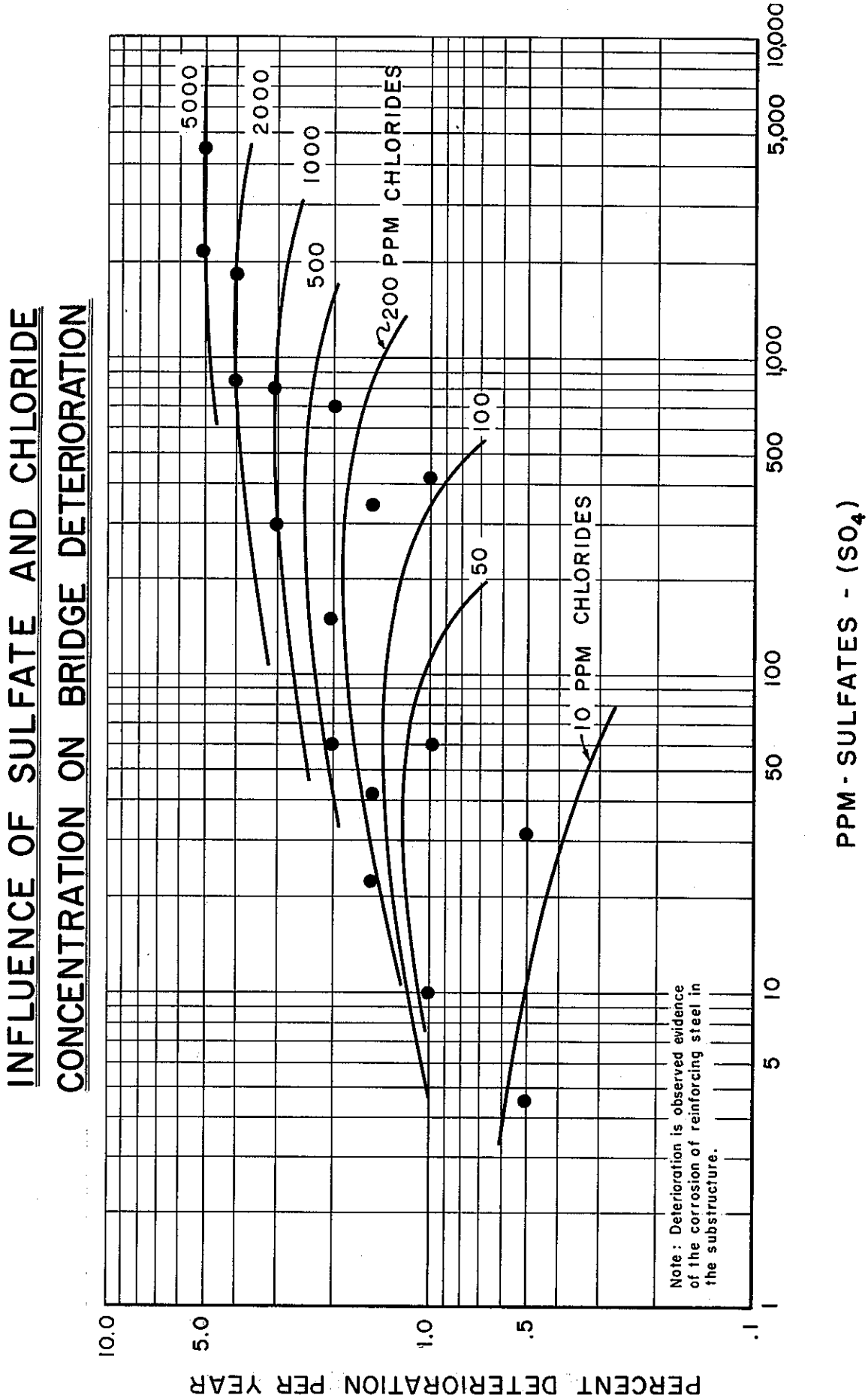

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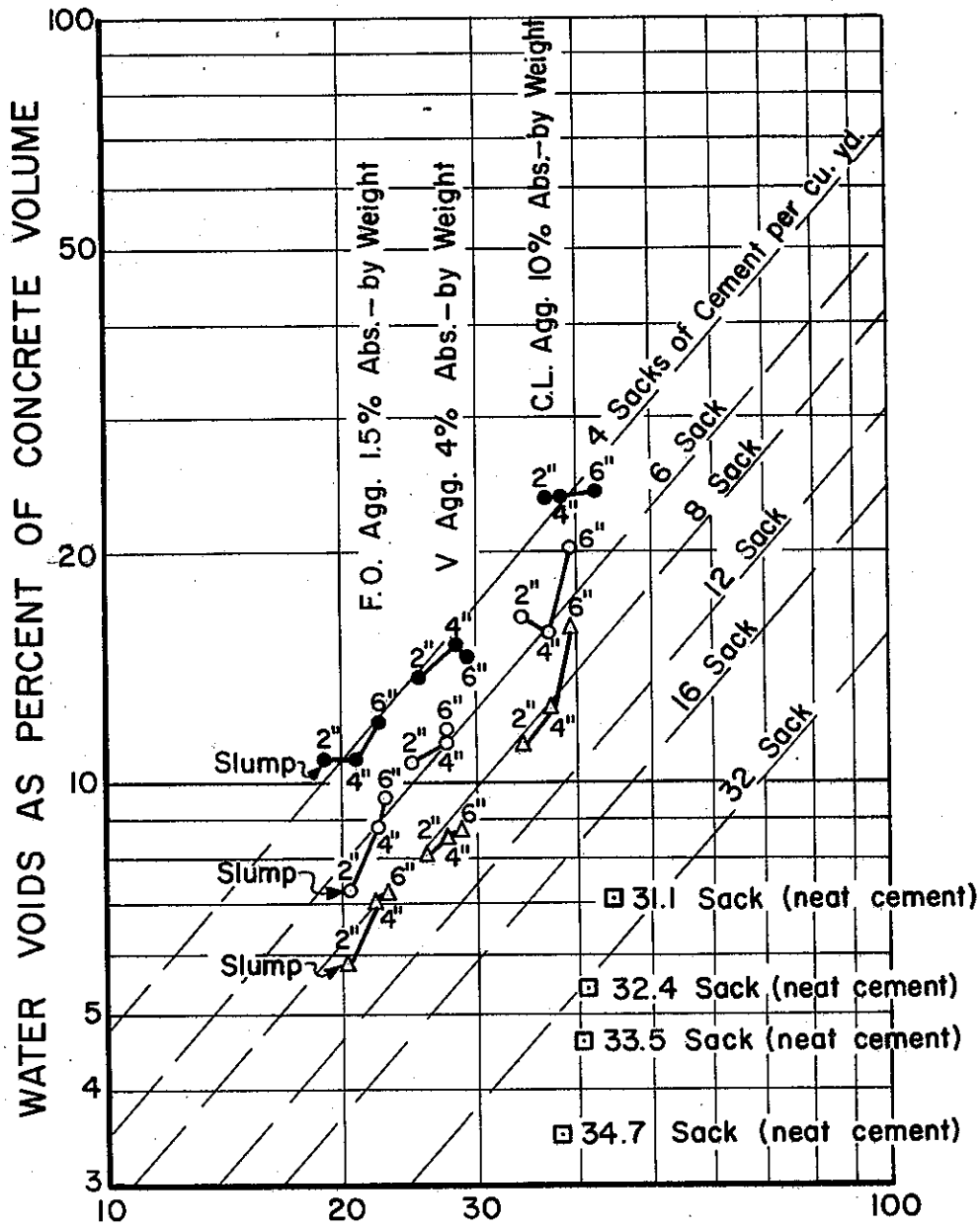
FIGURE 4

WATER VOIDS VERSUS MIXING WATER IN MATURE* CONCRETE



$$Vw = \frac{0.85\,(Wm)^{1.17}}{C^{0.717}}$$

Vw = Water voids as percent of concrete volume
Wm = Mixing water in percent of concrete volume
C = Sacks of cement per cubic yard

Note: Mixing water includes that contained by aggregate. Water voids computed from the volume of moisture gained and lost during prolonged exposure to 73.4° F. at 50% R.H. and 73.4 F. at 100% R.H.

* *Concrete aged in excess of 1 year*

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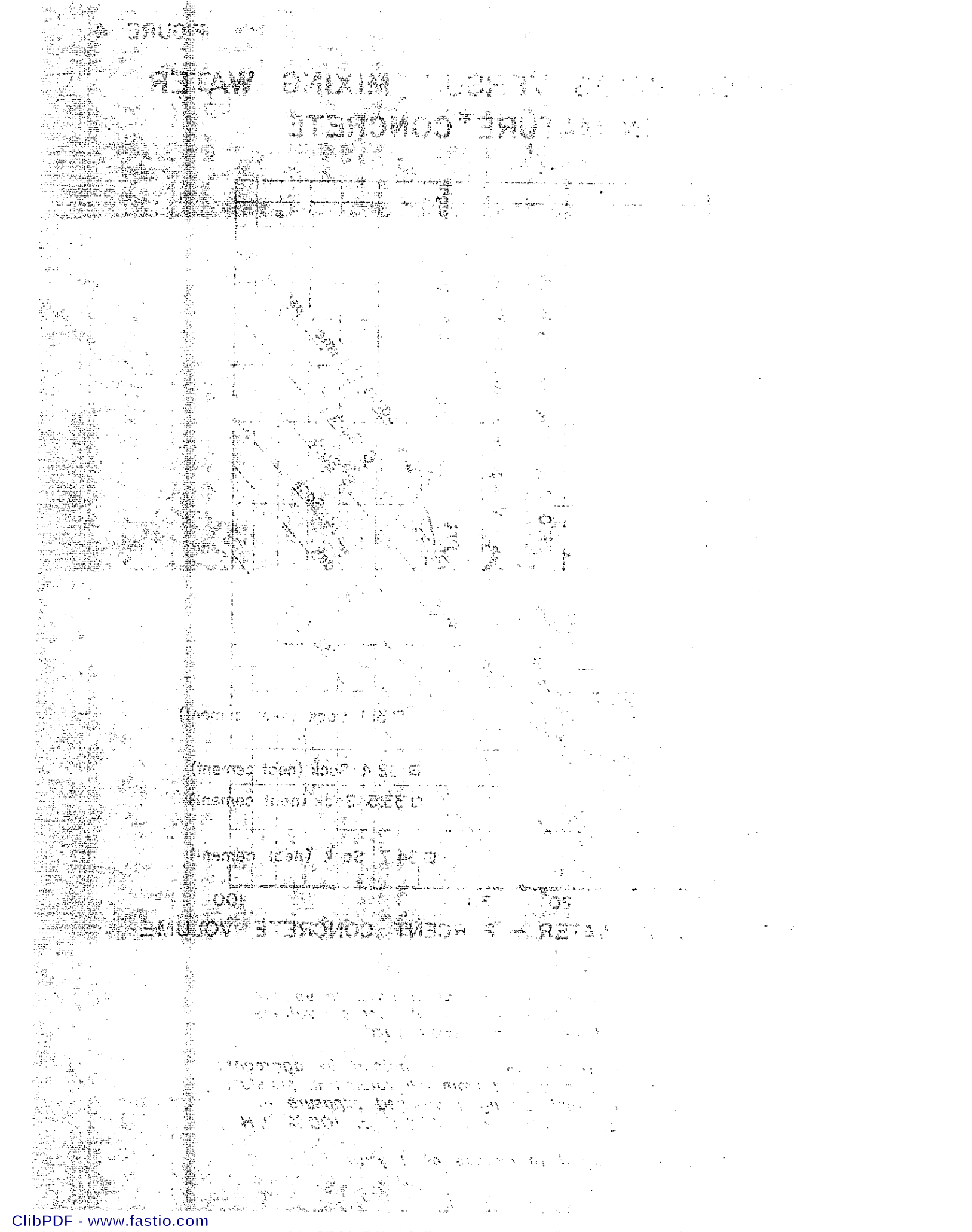

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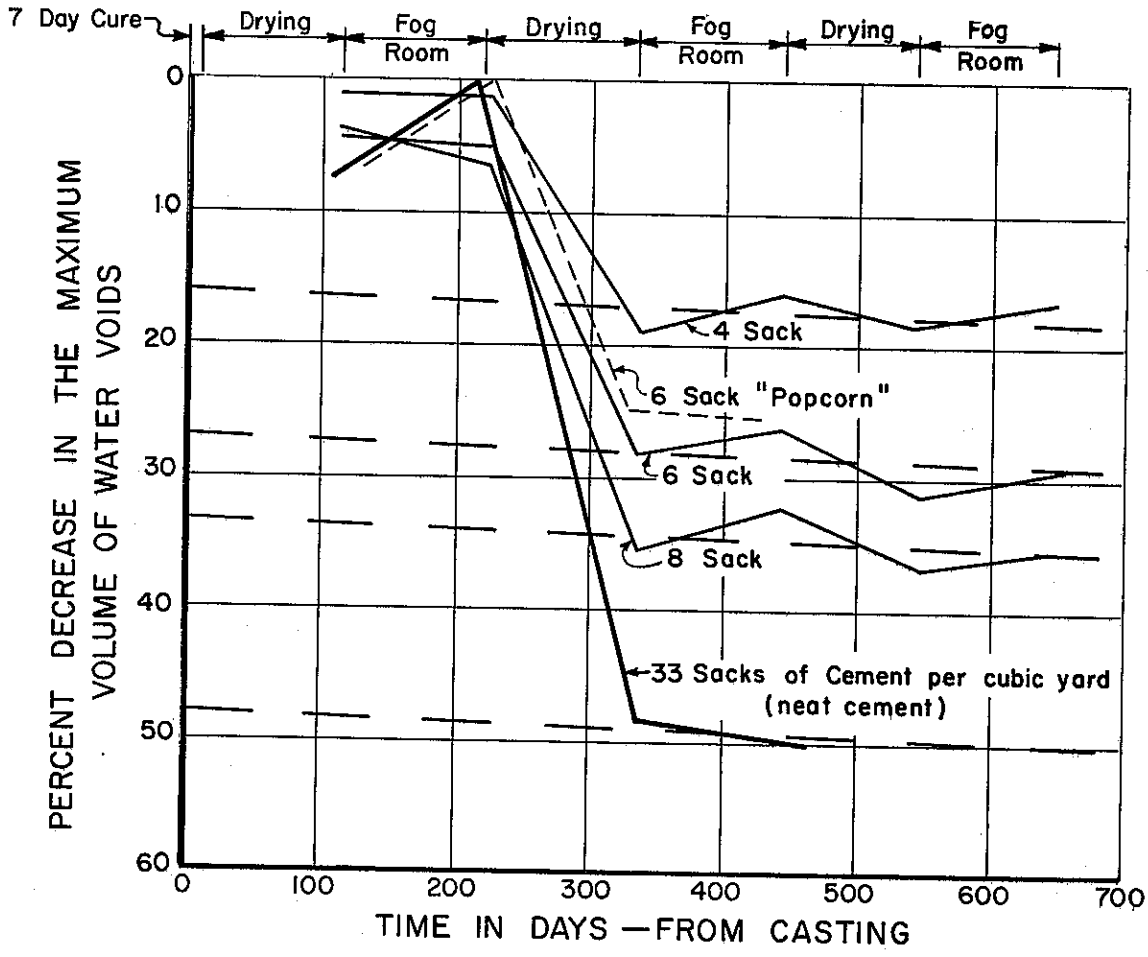
FIGURE 5

INFLUENCE OF CEMENT CONTENT ON THE VOLUME CHANGE OF WATER VOIDS— AGGREGATE CONSTANT



Note: Water voids computed from the measured volume of moisture gained and lost during indicated type of exposures. Concrete not oven dried. Dried at 73.4° F. and 50% R.H. Fog room at 73.4 F. and 100% R.H.

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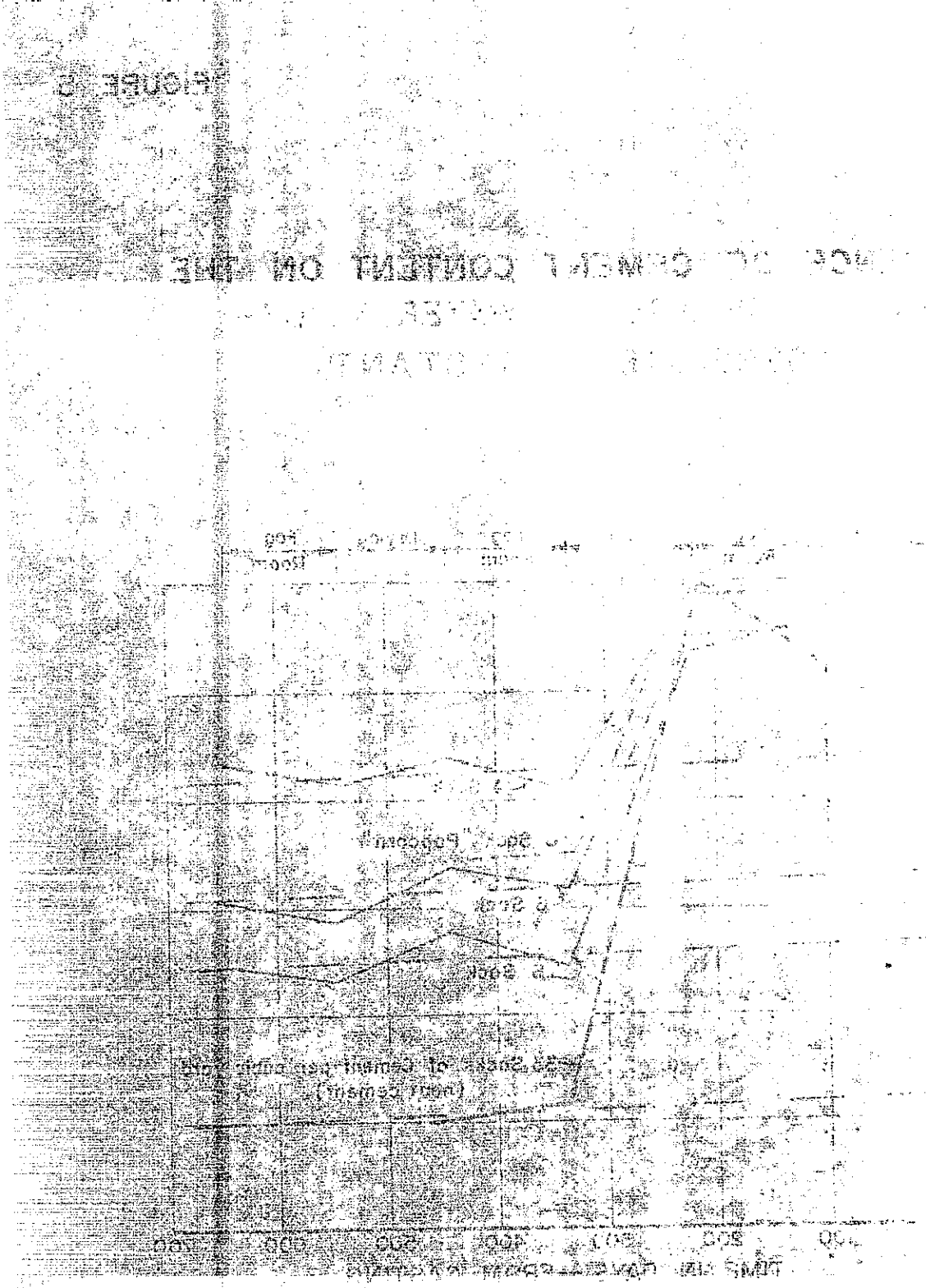

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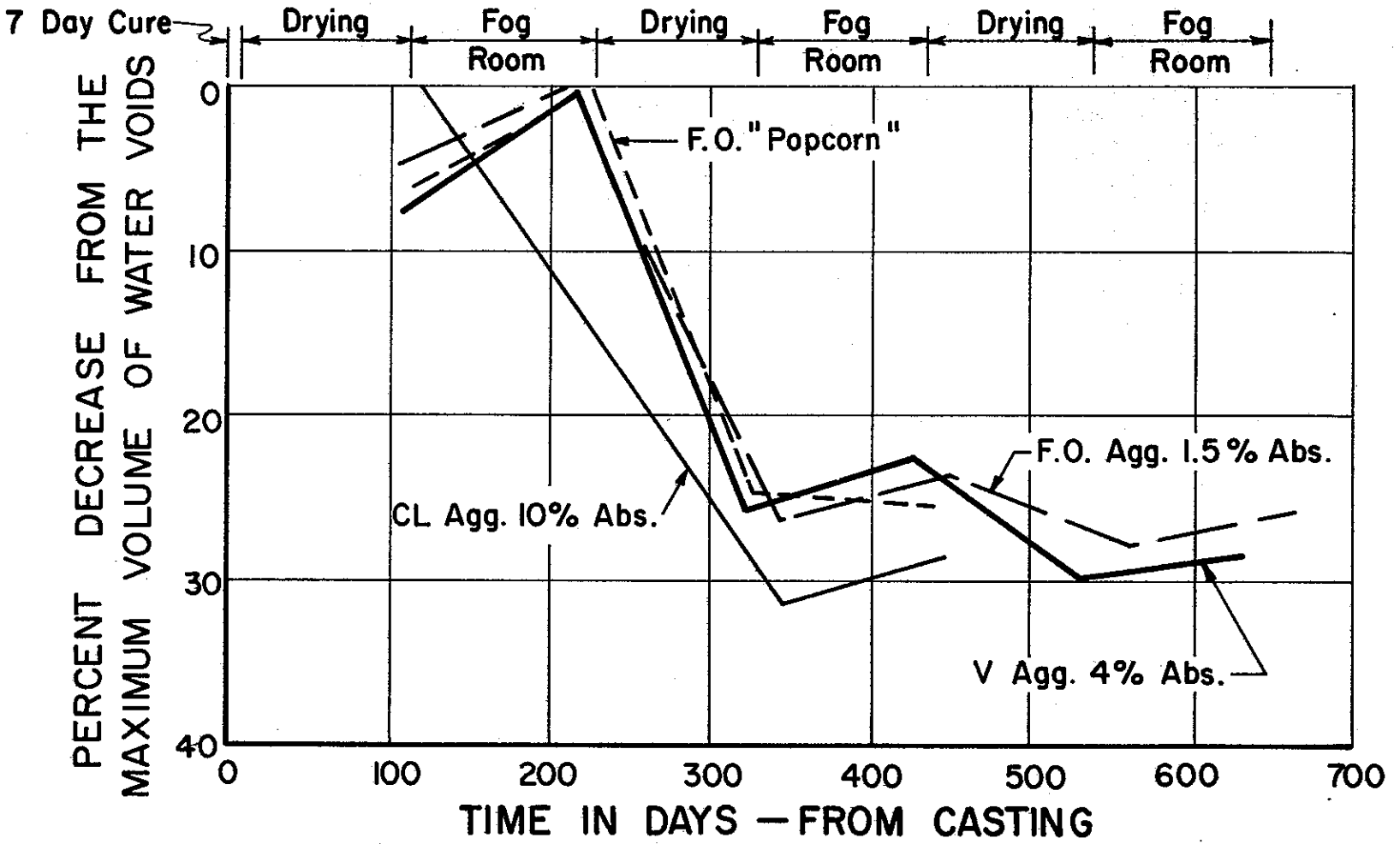
INFLUENCE OF AGGREGATE ON VOLUME CHANGE OF WATER VOIDS - CEMENT CONTENT CONSTANT



Note: Water voids computed from the measured volume of moisture gained and lost during the indicated type of exposures. Concrete not oven dried. Drying at 73.4°F. and 50% R.H. Fog room at 73.4°F. and 100% R.H.

FIGURE 6

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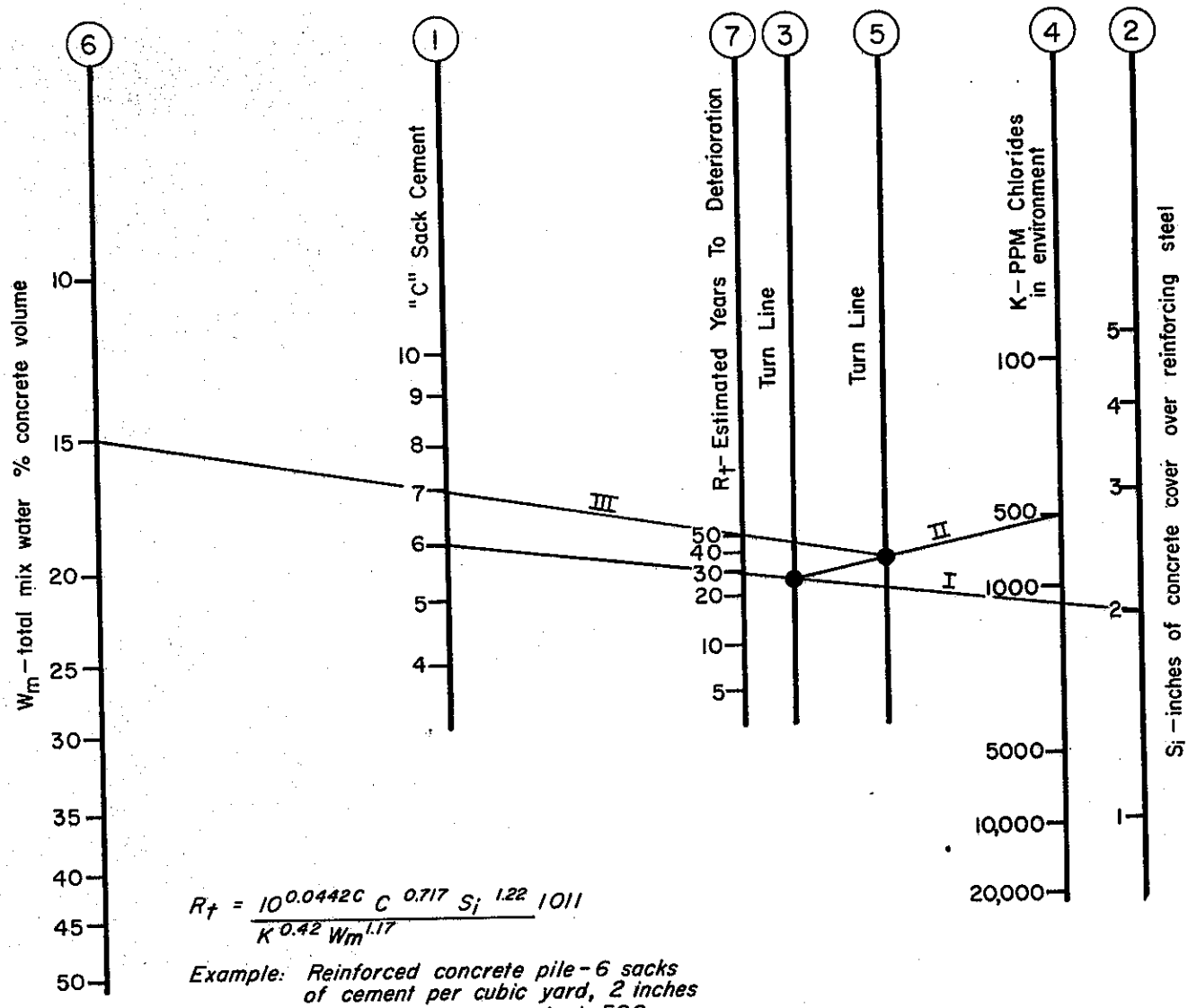
FIGURE 7

CHART FOR ESTIMATING DETERIORATION TIME FOR REINFORCED CONCRETE PILE



$$R_f = \frac{10^{0.0442C}\, C^{0.717}\, S_i^{1.22}\, 1011}{K^{0.42}\, W_m^{1.17}}$$

Example: *Reinforced concrete pile – 6 sacks of cement per cubic yard, 2 inches concrete cover over steel, 500 ppm chlorides in the environment, 15% by volume of total mixing water including that contained in the aggregate. Follow nomograph in sequence indicated by lines I, II and III. Answer is approximately 50 years.*

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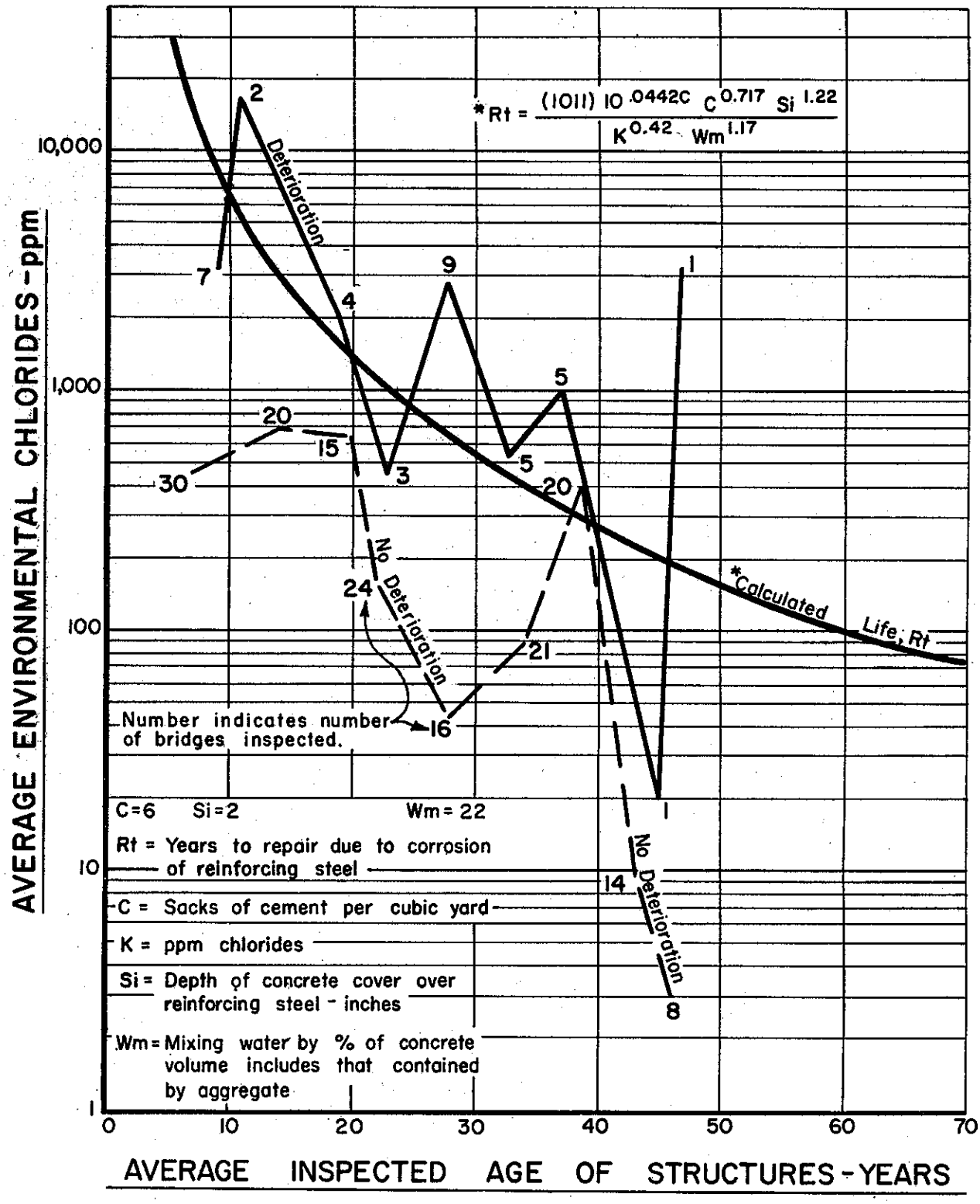
FIGURE 8

CONDITION OF 205 BRIDGES VERSUS AVERAGE AGE AT INSPECTION AND CHLORIDE CONCENTRATION



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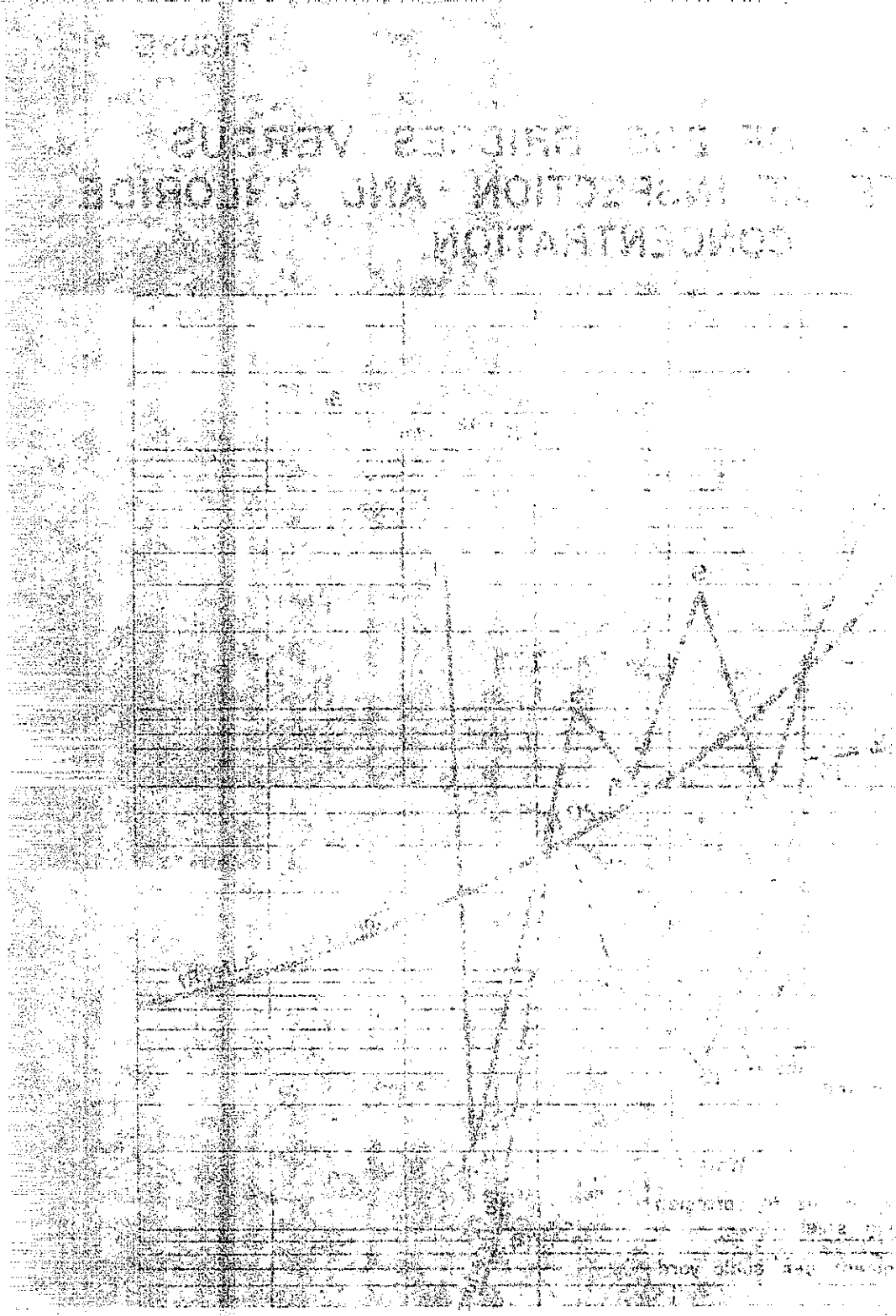

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Photograph 1

Vertical concrete cracking of 10 year old reinforced concrete bridge piles in tidal water near San Diego, California. Photo taken in 1962.

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Photo taken in 1963.

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Photograph 2

Spalling of concrete in beam caused by the corrosion of reinforcing steel, San Francisco-Oakland Bay Bridge. Photo taken in 1962.

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