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16. ABSTRACT

This is the third report of a series dealing with the causes of corrosion of reinforcing steel in the San Mateo-Hayward Bridge. This report, Part III, presents additional data on the causes of corrosion and also makes recommendations as to experimental methods of controlling corrosion.

Thirteen spans were selected for field and laboratory testing. These spans are believed to be representative of the reinforced concrete portion of the structure as a whole.

With respect to the concrete of the bridge, considered separately from the reinforcement, the studies indicate that the concrete has suffered general deterioration from sulfate attack occasioned by sea salts which have been absorbed. The present rate of deterioration due to sulfate attack cannot be estimated from the data now at hand.

Corrosion of the reinforcing steel, while arising in the main form the absorption of sea salts by the concrete, is not dependent on deterioration of the concrete by sulfate attack. The main cause of the corrosion of the reinforcement is the non-uniform distribution of sea salts which sets up corrosion cells of the differential concentration type. The activity of these cells is believed to be maintained by the porous character of the concrete and its relatively high level of contained moisture.

The studies indicate that the present method of repair, consisting of removal of affected concrete, sand blasting and replacement with shotcrete, does not offer assurance of permanence.

It is recommended that two possible methods of controling corrosion be undertaken on an experimental basis.

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STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

CAUSES OF CORROSION OF REINFORCING STEEL

in

SAN MATEO-HAYWARD BRIDGE

IV-S.M,Ala-105-B,A

PART III

66-04

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

April 20, 1956

IV-S.M,Ala-105-B,A
Lab. W.O. No. 5018-R-98

Mr. F. W. Panhorst
Assistant State Highway Engineer
Division of Highways
Sacramento, California

Dear Sir:

Submitted for your consideration is:

PART III

of a report on

CAUSES OF CORROSION OF REINFORCING STEEL

in

SAN MATEO-HAYWARD BRIDGE

IV-S.M,Ala-105-B,A

Study made by . . . . . Technical and Structural Materials Sections
Under general direction of. . . . . Bailey Tremper and J. L. Beaton
Report prepared by. . . . . Bailey Tremper and R. F. Stratfull

Very truly yours

F. N. Hveem

F. N. Hveem
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M.W.Gewertz

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SYNOPSIS

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CAUSES OF CORROSION OF REINFORCING STEEL IN SAN MATEO-HAYWARD BRIDGE

PART III

INTRODUCTION

This report describes the investigations made during the year 1955 of the causes of the corrosion of reinforcing steel in the San Mateo-Hayward Bridge. Two previous reports, Parts I and II, described the preliminary investigations which led to the adoption of the program for the 1955 investigations.

Parts I and II presented data showing evidence of deterioration of the concrete itself through sulphate attack and also outlined the probable mechanism which was responsible for the corrosion of the reinforcement and the subsequent cracking of the concrete. These preliminary findings reported in Parts I and II were based on the test results obtained on a limited number of spans. It could not be assumed that the conditions found on the limited number of test spans would be representative of the structure as a whole.

The objectives sought in the 1955 investigation were as follows:

(1) To obtain sufficient test data to fairly represent the present condition of the structure from end to end.

(2) To identify the locations of each test so that future tests at the same locations will definitely establish any physical or electrical changes in the structure.

(3) To use new techniques for measuring physical and electrical properties of the concrete, and of the structural system of concrete plus reinforcement.

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(4) To initiate a laboratory program to investigate the fundamental factors which influence the corrosion of the reinforcing steel.

(5) To investigate various means of protecting existing reinforcement against further corrosion; i.e. by organic or inorganic coatings applied to the steel in the areas of concrete repair.

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OUTLINE OF TEST PROCEDURE

In order to obtain data representative of the structure as a whole, it was necessary to select a sufficient number of spans spaced at intervals over the length of the structure. The bridge consists of 1176 spans numbered from 24 at the west end to 1199 at the east end. Originally it was planned to conduct tests on 20 spans spaced at intervals of about 60 spans. However, limitation of time and funds made it necessary to reduce the program to 13 test spans. Figure 1 is a schematic layout of the bridge indicating the sections that have been repaired under State ownership and the parts of the spans to which a bituminous coating had been applied previously. It is believed that the selection of spans for study is sufficiently broad to include the major variables in design, location and type of previous repair work. Proportionately more spans were selected from the westerly portion of the structure because of a greater variety of conditions and more advanced deterioration.

The State-owned Toll Bridge District designed and furnished portable scaffolding which was set up at each test span for the use of the laboratory crews who made the tests. The scaffolding permitted access to the complete underside of a span and, except in those spans of 35-foot length, both sides of one of the bents in each span. One day each week was required to move the scaffolding to a new location. Four working days were available each week for tests in each span. The field investigation was started on July 1, 1955 at span 70 and was terminated at span 1165 on November 18, 1955.

The field investigation was divided into two parts. The first part was performed by a crew of two that made electrical measurements on the surface of the concrete. Electrical surface potential measurements were made on a closely spaced grid over all of the exposed underside of the selected span in one-half of the roadway width. Frequent measurements were made of the resistivity of the concrete and of the relative humidity of the air beneath the bridge. Areas of repair and all visible cracks were mapped in the section of the span under investigation. Equipotential contours of the area surveyed were plotted and areas of cathodic and anodic polarity were determined. The latter information was used to select locations for core drilling. The work described above required two working days.

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A second crew of three worked the remaining two days of the week. This crew used the soniscope to determine the pulse velocity of the concrete, generally over the same pattern as that employed by the crew on electrical measurements. Pulse velocities were at times also measured in the opposite half of the roadway width. Four 4" x 8" cores were taken in each span; two from the deck and two from the beams. Rebound number by the Swiss hammer was measured in a few spans. This crew also independently mapped all areas of repair and all visible cracks in the portion of the span under investigation.

Laboratory work was performed on the drilled cores as they were furnished by the field crew. The cores from the first two spans (No. 70 and No. 129) were soaked in water for 7 days and then tested for pulse velocity, static modulus of elasticity and compressive strength. Care was taken to discontinue loading of the core as soon as its ultimate strength was reached and before the specimen was ruptured. It was then sawed into discs one inch thick. These discs were first air dried and then individually broken down to separate mortar from coarse aggregate. The mortar was then pulverized for chemical analysis. Tests were made to determine the amount of salts leached out of the concrete during soaking in the laboratory. The amount of chlorides lost did not exceed 0.005 per cent by weight of the mortar contained in the core. The error occasioned by soaking before analysis, although not important, was avoided in subsequent cores from the beams, by sawing off the outer 1 inch for analysis before the core was soaked. Chemical analyses were not made on the remaining portion of the beam core. Because the chloride content of the deck was relatively low, these cores were analysed after soaking and physical testing and the entire core was treated as a single sample.

Samples of pulverized air-dry mortar were analysed for chloride-ion (herein designated as "chlorides"), sulfuric anhydride (SO_3), magnesia (MgO) and calcium oxide (CaO) in the portion soluble in dilute hydrochloric acid. The contents of SO_3 and MgO were calculated as percentages of the original cement on the assumption that it contained 64 percent CaO.

A sample of aggregate from the pit used in the construction of the bridge was made into concrete. After curing, it was pulverized and analysed in the same way as the bridge cores. The calculated percentage of MgO found in this test was very nearly the same as that in the cement used. This demonstrated that the percentages of MgO found in the cores were not affected by analytical errors.

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TEST DATA

The results of all of the field tests and the locations at which they were made are on file in the Materials Laboratory. Charts showing the equipotential contours in each span have been prepared and are on file. The charts for three representative spans are included in this report and are shown as Figures 2, 3 and 4. The charts also show the measured resistivity of the concrete, the areas that have been repaired and the location of visible cracks in the concrete.

In this report the test values of pulse velocity are shown as averages only for each span. The individual results do not have meaning at the present time because there are no previous test results at the same location to which they can be referred. They will of course, have value in future years when compared to subsequent readings to indicate changes in the condition of the concrete at particular points in the structure.

Much of the test data is summarized in Tables I, II and III. Other tables and figures are used to illustrate points made in the discussion and will be referred to in the succeeding text. A complete list of tables and figures is given on the page which follows the synopsis of this report.

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DISCUSSION

Part I offered evidence that the concrete in the one span investigated was not of high quality and probably had suffered deterioration since its construction as a result of sulphate attack. The data presented in Tables I and II of this report show that the concrete throughout the structure has low strength and high absorption. The probability that a general sulphate attack has occurred throughout the structure is indicated by the rather uniform content of magnesia (MgO). Data on the static modulus of elasticity (2,500,000 average) show the concrete to be low in this property. This low modulus of elasticity is an indication of deterioration of the concrete. (Concrete of even mediocre quality could be expected to have a modulus of at least 4,000,000 at the age of this structure if not affected by deterioration.) The pulse velocities are significant at present only to the extent that they indicate a lower present quality of concrete in some spans than in others. The evidence as a whole is quite positive that the concrete has undergone deterioration as a result of sulphate attack. The rate of deterioration can be estimated only by comparing presently available data with the results of future tests.

The effect of sulphate attack and the resultant concrete deterioration on the progress of corrosion of the reinforcement is not clear. The indicated loss of strength in the concrete does not seem to be directly related to the development of the concrete cracking due to rust formation. The pressures that the rust can develop are undoubtedly sufficient to crack much stronger concrete. The development of corrosion in the reinforcement is related to, but is not dependent on, sulphate attack of the concrete because the salts could be absorbed without necessarily causing sulphate attack.

Figures 2, 3 and 4 show equipotential contours in the surface of three spans. They are characteristic of the electrical and physical conditions found in the other spans. The charts present definite evidence of the existence of anodic and cathodic areas within the span and indicate the electrical potential between these areas.

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Figure 2 represents the conditions found in span 70. About 60 percent of the bottom on one beam was repaired with shotcrete in 1951. Cracks have appeared in the remainder of this beam and also in the adjacent beam. These cracks presumably occurred since the beams were repaired. The equipotential contours indicate the measured electrical potentials. The contours shown in red close around centers of anodic areas (plus) and the green contours close around cathodic areas (minus). The corrosion of the steel occurs in the anodic areas. It will be noted that the steel in repaired areas is now cathodic to the steel in the original concrete. Presumably the steel that is now encased in shotcrete was in an anodic area before the repair. Anodic areas are present at locations adjacent to the shotcrete and elsewhere in the beams where cracking has occurred. Two anodic areas have developed in the deck slab and may be the result of creating cathodic areas with shotcrete or the loss of the previously anodic location. Anodes are present in the caps and piles including portions that have been repaired with shotcrete. The average resistivity of the concrete in the anodic areas is 30,000 ohm-cm and in the cathodic areas, 54,000 ohm-cm.

Figure 3 represents span 163 which is adjacent to the steel lift span section. Span 163 is at a higher elevation above the bay than other concrete spans. The deck unit is free from longitudinal cracking although there are faint or embryo anodes starting at the ends of one beam. Otherwise, no strong corrosion cells are indicated by the electrical measurements in this deck unit. There are strong anodes in the diaphrams and caps and cracks in the concrete were observed in one diaphram. The resistivity, measured in cathodic areas averages 76,000 ohm-cm.

Figure 4 depicts span 553 which is representative of a badly deteriorated span that has not been repaired with shotcrete. The field crews observed longitudinal cracks in the concrete covering approximately 50 percent of the length of the underside of the beams. There are strong anodes where cracking has occurred, but elsewhere the beams are neutral or only weakly anodic. The entire deck slab is cathodic. The anodes in the caps appear to be related to the cathodic areas in the diaphragms. The resistivity in anodic areas averages 37,000 ohm-cm and in the cathodic areas, 50,000 ohm-cm.

The algebraic difference in potential between anode and cathode represents the driving force of the corrosion cell. In the spans illustrated, surface potentials in the deck unit indicate that a maximum surface driving force of 0.4 volts was responsible for the corrosion of the reinforcing steel.

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L.A. Woodworth (reference 1) has constructed differential concentration cells consisting of steel plates embedded in blocks of concrete containing sea salt of varying concentration. Corrosion cells were established by electrically connecting two blocks. A potential and flow of current was observed whenever there was a difference in the concentration of salt in the two halves of the cell. Potentials up to 0.35 volt were measured. They were thus of the same order of magnitude as those measured on the San Mateo-Hayward Bridge.

The maximum potential that can be generated by such cells probably does not exceed 0.5 volt. Under relatively constant potentials the resistivity of the concrete should be the determining factor in the activity of the cells.

In Table IV the spans have been arranged in three groups according to the degree of deterioration. The classification is on the basis of the linear feet of longitudinal cracking along the bottom of the beams, expressed as the percentage of the total length of the beams investigated in the span. Span 410 has been omitted because it was repaired shortly before the survey and there is some uncertainty as to the actual amount of cracking that existed prior to repair. The last column of Table IV shows the resistivity of the concrete in anodic areas. It will be noted that decreasing resistivity is accompanied by increased cracking; also that when the resistivity exceeds about 60,000 ohm-cm there has been little or no cracking.

Figure 5 is a plot of deterioration against resistivity at anodic areas. The general trend is that increasing resistivity is accompanied by decreasing deterioration. Resistivity of 60,000 ohm-cm is apparently required to inhibit or prevent the start of accelerated corrosion in this concrete.

The quantity of reinforcing steel corroded is a function of the amount of current (ampere-hours for example) flowing through the corrosion cell. The rate of current flow cannot be accurately measured in this structure but can be represented approximately by the potential difference between anodes and cathodes divided by the resistivity. Figure 6 is a plot of deterioration in the several spans against the ratio $\frac{\text{potential}}{\text{resistivity}}$ which for convenience, has been designated as "cell ratio". The value of potential used in the cell ratio is the maximum voltage differential between the cathode (which is generally in the deck) and the anode (which is generally in a beam). The resistivity value of the concrete used in this ratio was measured at anodic areas. The curve

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indicates that there is a definite relationship between the cell ratio and deterioration of the concrete. The curve was determined by the corrosion cells in the deck and beams in the spans under consideration and the shape of the curve may be representative of the deterioration of an average corrosion cell in the bridge. As the curve was drawn from average values, it does not necessarily represent any single cell, but may represent the combined effect of all the cells in a particular span.

Figure 7 is similar to Figure 6 but consideration is given to each plotted point as to whether or not a crack has developed in the concrete. Each point is plotted on the curve according to its cell ratio value. On this chart the circles indicate that there is no crack in the concrete. The squares show that there is a crack. The plotted points do not indicate any particular degree of deterioration other than the presence or absence of cracks in the concrete. It is interesting to note that the division between cracked and uncracked concrete on the curve is about at the 20 percent deterioration point and that the cell ratio at this point is about 0.5. This value is representative of present conditions. It is conceivable that in time cracking will occur under lower cell ratios.

In some of the spans potentials were measured by direct connection to the reinforcing steel. In others the measurements were related to a fixed potential point on the surface of the concrete. In the later case, differences in potential were measured and the contours were plotted. The observed potential relative to steel was not known when this method was used. Five of the spans were in the first category and Figure 8 is presented to illustrate the relationships obtained in these spans. The crosses in this figure represent the apparent neutral 1/2 cell potential of the steel to a copper sulphate reference cell. The sloping line represents the neutral potential of the steel for varying amounts of resistivity (specific electrical resistance of the concrete) as determined by the position of the crosses. Cathodic potentials fall to the left of the curve and anodic potentials to the right. Plotted circles represent cathodic potentials. Triangles are anodic potentials at locations where the concrete was not cracked and squares, anodic potentials in cracked concrete.

The chart shows that cracking in these spans occurred only when the anode resistivity was below 45,000 ohm-cm. but that there are two points where the resistivity was this low but in uncracked concrete. The difference in behavior seems to be related to the excess in the negative potential from the neutral point of the steel. A chart of this kind may be useful in indicating the needed potential for cathodic protection when applied to each corrosion cell.

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This chart also indicates that corrosion cells may be set up when reinforcing steel is embedded in concrete of varying resistivity. This is indicated by the fact that in the plotted cells the average resistivity of the concrete in cathodic areas is 64,000 ohm-cm. and only 42,000 ohm-cm. in anodic areas.

The resistivity of the shotcrete now in place in the structure, as determined by the average of all measurements made, is 60,000 ohm-cm. On the other hand, the resistivity of newly placed shotcrete has been measured at 150,000 ohm-cm. The drop in resistivity with age may explain why cracking has developed in some of the older shotcrete.

It has been shown that the resistivity of the concrete is a critical factor in determining the rate of corrosion. It is of value therefore, to consider the factors that may lower the resistivity. Salt is certainly important. Consider a small unit volume of concrete, the pores of which are filled with water containing dissolved salt. In dilute solutions, resistivity should decrease in proportion to the concentration of salt. Now assume that through evaporation, one-half of the absorbed water has been removed. Although the concentration of salt in the remaining water has doubled, the resistivity of the concrete should not be materially changed because there is only one-half as much water to serve as a conductor. In other words, the number of ions available from the salt to transmit current has not been altered appreciably. There are evidently factors connected with moisture change other than the quantity of salt, that influence resistance. That is to say, that electrical current is conducted through the concrete in ways other than by salt ions. It was shown in Part II that the resistivity of air-dry concrete was from 10 to 30 times that of the same concrete when moist or wet.

The prevailing level of moisture in the concrete, then, is an important factor that influences the activity of cells and the consequent rate of rusting of the steel. It has been observed that the deterioration rate of the structure appears to be less in the easterly end of the San Mateo-Hayward Bridge. This could be the result of a difference in the moisture level in the concrete at the two ends of the structure. It has been reported that fog is more prevalent toward the west or San Mateo shore of the bay and this condition could result in a higher degree of saturation of the concrete.

Differences in relative saturation could explain the observed differences in resistivity and rate of corrosion at the two ends of the structure.

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Span 163 is adjacent to the steel lift span section and is at a higher elevation above the water than is the major portion of the bridge. There is no cracking in the deck unit of this span and the resistivity at the anodic areas was measured at 67,000 ohm-cm. which is the highest value measured in an anodic area in the structure. The span is located near the west or San Mateo end of the bridge where corrosion is most prevalent. It seems probable that its higher elevation above water has resulted in a lower level of contained moisture, thus accounting, partially at least, for its higher resistivity.

The relative humidity of the air beneath a particular span was measured at intervals during the days that the laboratory crews were working at that location. Average results of humidity observations are shown in Figure 9. Two curves are plotted, one for measurements during July, August and September in spans 70 to 560 at the west end, and the other for measurements during October at spans 925 to 1165 at the east end. The average relative humidity at the east end is 9 percentage points lower than that at the west end.

A differential of 9 percent in relative humidity of the atmosphere is of considerable importance in establishing the moisture level of concrete. For example, at a given temperature, the rate of evaporation of water into air of 75 and 84 percent relative humidity is in the proportion of:

$$\frac{100 - 75}{100 - 84} = 1.55$$

The rate of evaporation into the drier air is therefore, more than 50 percent greater.

The differential in relative humidity observed at the bridge site may be simply a reflection of the difference in time of year at which the two groups of measurements were made. There is evidence however, in the records of the U.S. Weather Bureau that the differential is sustained.

The weather bureau stations closest to the San Mateo-Hayward Bridge at which relative humidity is recorded are located at San Francisco Airport and Oakland Airport. At these stations, average monthly relative humidities for the period of interest were as follows:

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Relative Humidity at Airport Stations

Month (1955)	4:30 AM		10:30 AM		4:30 PM		10:30 PM	
	S.F.	Oak.	S.F.	Oak.	S.F.	Oak.	S.F.	Oak.
July	91	89	67	69	67	63	87	84
August	89	88	65	70	63	60	86	84
September	93	88	66	68	65	60	87	83
October	90	85	70	69	65	59	84	81

The data support the belief that the differential between the two ends of the bridge as observed at the time of measurement would have prevailed had the observations been made simultaneously.

U.S. Weather Bureau summaries for a 20-year period show the following average relative humidities:

Relative Humidity at Airport Stations
(average for 20-year periods)

Time	San Francisco	Oakland
4:30 AM	88	86
10:30 AM	71	69
4:30 PM	68	63
10:30 PM	84	81

The data indicate the probability that the relative humidity at the east end of the bridge is consistently lower than that at the west end.

In any event, the relative humidity under the bridge as a whole is high and it seems certain that the level of moisture saturation within the concrete must also be high.

Because of the apparent importance of moisture and its effect on resistivity, field tests to determine the relative saturation of the concrete at different locations in the structure should be informative. Such tests could be made by inserting humidity sensing elements into small drilled holes in the concrete.

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The effect of salt content (chlorides) on resistivity is not well established by the test data at hand. The beam cores in which chlorides were determined were taken at points about 10 inches above the lower face and were not at the locations of cracked or severly anodic concrete. The percentage of chlorides as shown in Tables I and II, as previously reported in Part I have been found to vary over a considerable range, even in adjacent sections from the same core. Figure 10 shows the variation in chloride content with depth from the surface of the member as determined in cores from two spans.

The importance of salt in influencing resistivity must be recognized but the available data do not demonstrate a correlation between salt content and resistivity.

The presence of absorbed salt makes it possible for differential concentration cells to be established. Such cells, it will be recalled, are set up when two pieces of metal electrically connected, or two parts of a single piece of metal, are embedded in connected electrolytes of different salt concentration. Differences in salt concentration in the concrete of the bridge are shown by the test data. Also at locations of equal salt content there may be differences in concentration in the contained water because of differences in the amount of water present. Even though the salt were at some time evenly distributed throughout the concrete, the forces of capillarity and evaporation would tend to concentrate it in some regions and deplete it in others.

The experiments of Woodworth (1), previously referred to, demonstrate that differential concentration of salts in concrete can give rise to very active corrosion cells.

Other types of cells that may be set up in the structure are those of differential aeration, differential heating and differential stress and strain. The magnitude or the contribution of such types of corrosion cells to the over-all corrosion effect is not known but at present it is believed that the differential heating and differential stress and strain types are minor in comparison to those of differential concentration and probably to differential aeration.

Corrosion cells will not remain active for long if access of oxygen to the cathode is prevented. In the absence of oxygen the cell soon becomes polarized by a build up of hydrogen at the cathode. In other words, oxygen is required to unite chemically with the accumulated hydrogen at the cathode to depolarize the cell and maintain its activity. Woodworth (1) demonstrated that an active corrosion cell of steel and concrete became passive when access of oxygen was prevented by burying the cell 6 feet in the ground. He

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also immersed active cells in water and found that they became dead in two months but could be revived upon removal from the water and drying.

In Part I, it was shown that the concrete in the San Mateo-Hayward Bridge is unusually permeable to air. This fact may be responsible for the continued activity of the corrosion cells in the bridge.

In this report considerable space has been devoted to a statement of the apparent conditions that have caused corrosion of the reinforcement in the bridge. The better these conditions are understood, the greater the probability of devising successful corrective measures. The investigations could be used with profit in devising means of guarding against a similar result in new construction. The problem at hand however, is to determine means of eliminating, or at least delaying, the corrosion that is now taking place in the existing bridge. This subject is discussed in the succeeding section.

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POSSIBLE METHODS OF STOPPING CORROSION

Among the possible means of inactivating the differential concentration cells that apparently are the main causes of corrosion, are the following:

1. Remove the affected concrete, sandblast the reinforcement, and replace with fresh concrete such as shotcrete.
2. Combine (1) with a dielectric coating of the exposed reinforcement that will not interfere with the bond between concrete and steel.
3. Equalize the salt content of the concrete.
4. Exclude moisture from the concrete.
5. Inject an inhibitive solution into the concrete.
6. Exclude oxygen from the reinforcement.
7. Install cathodic protection.

Method (1) is currently being used on the bridge. There is evidence indicated by the corrosion of steel within some of the older shotcrete that this treatment is not necessarily permanent and possibly is detrimental. Although the quality of protection afforded by ideal shotcrete should be excellent, it is difficult to apply it without producing lenses of rebound or otherwise porous material. Such non-uniformity can be productive of differential concentration or aeration cells. Even though the shotcrete affords perfect protection to the steel it encases, its application immediately sets up new conditions within the span that has been partially repaired. The shotcreted section can become cathodic to adjacent areas and cause portions that formerly were neutral or cathodic to become anodes. This action appears to have occurred in span 70 (see Figure 2 and the accompanying discussion).

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Method 2. Laboratory experiments made to date indicate that applications of epoxy or vinyl formulations do not seriously affect the bond between the steel and the concrete if the concrete is placed while the coating is still tacky. Tests show that the passage of current from the steel to the concrete can be effectively prevented initially. Accelerated tests under an impressed current point to a possible long useful life of the coating. However, such coatings could be applied only when the reinforcement is exposed for repair. The repaired areas are made cathodic initially by the application of shotcrete and as long as they remain cathodic do not require additional protection against corrosion of the steel in that region. An effective coating on the steel would however, tend to exclude oxygen and thus would tend to render inactive any cell that might be set up in which the coated steel served as a cathode. A treatment of this nature however, does not get to the seat of the difficulty because the steel in the area adjacent to the repair has not been protected and it may become anodic, if not to the repaired and cathodic area, then to some other cathodic area within the unit. Corrosion currents from the unprotected steel near the repaired area might then be intensified. The over-all result from coating the steel with a dielectric probably would not be a marked improvement over the simple application of shotcrete.

Method 3. If the moisture within the concrete could be stabilized so that its further movement were prevented, the salt might in time become uniformly distributed and the differential concentration type of cell would then be eliminated. The application of a bituminous coating several years ago may have contributed to the stabilization of the moisture level. It is now evident however, that it did not result in a uniform distribution of the contained salt and that it did not permanently arrest corrosion. It seems probable that the membrane even though initially resistant to the passage of liquid water into the concrete, was not impervious to the passage of water vapor or air. Since the membrane could not be applied to separate the deck section from the caps and piles, capillary rise of water from the bay into the deck section could not be prevented. Other coatings might be more effective but there is no assurance that the salt within the structure would be equalized within a reasonable period of time.

Method 4. Silicone treatments are useful in preventing the absorption of liquid water while at the same time permitting the passage of water vapor. Under good drying conditions the concrete so treated may be expected to lose water. Under conditions of high relative humidity, such as exist at the bridge site, the equilibrium point at which

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vapor would pass into the structure at the same rate as out would be established with a relatively high moisture content in the concrete. The possibility of increasing resistivity sufficiently to arrest corrosion by this means seems to be remote.

Method 5 contemplates the injection into the concrete of a solution that would inhibit corrosion. No such solution of proven efficacy is known. Its uniform injection throughout the body of the concrete would appear to be difficult of accomplishment.

The foregoing discussion of methods 1 to 5, has been presented to bring out the difficulties that appear to work against their success. The next two methods to be described appear to offer more promise.

Method 6. It is possible that oxygen could be excluded from the structure by a suitable coating. It is evident, however, that the bituminous coating applied some years ago to the bridge, while possibly effective initially in retarding the passage of air, was not effective in stopping corrosion for more than a few years at the most. Thicker coatings renewed frequently might be effective. Enclosure in a metallic sheath is another possibility, probably very costly, and unless made of corrosion resistant metal the maintenance costs would also be high.

It is believed that the effectiveness of an oxygen-excluding treatment is seriously reduced if it is applied to the surface of the concrete unit as a whole. It would be much more effective if applied to cathodic areas only leaving the anodic areas untreated. This is because the exclusion of oxygen from the cathode but without restriction of its access to the anode favors the establishment of a differential aeration cell having a polarity opposite to that of the present corrosion cell. Insofar as the deck units are concerned the cathodic area is generally in the slab. Since the top surface in general now has a bituminous mat the treatment would need to be applied to a relatively small area consisting of the slab edges and that portion of the bottom surface that is not occupied by the beam stems. The beam stems should have any existing coating removed so as to permit as much aeration and drying as possible. The deck itself should be kept as nearly saturated as possible as a further means of excluding oxygen. Prior to coating, it might be practicable to attach half round metal conduits filled with porous material to the lower face of the slab into which water would be fed.

Method 7. Cathodic protection has been used effectively in pipe lines buried in the ground. While there is little information available on the protection of steel in concrete

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by this method there appear to be no basic or theoretical reasons why it would not be effective in concrete. Study and experimentation is needed to determine the best method of application. A trial installation on a small portion of the bridge should develop sufficient information to indicate the cost and effectiveness of a complete installation.

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RECOMMENDATIONS

1. A new field survey should be conducted in 1957 to determine if changes have taken place in the selected spans since 1955. The survey would be conducted along the lines of the 1955 investigation but could be made at less cost. Changes in the quality of the concrete itself and in the extent of corrosion would be evaluated. Such a study would be of assistance in estimating the probable future service life of the structure.

2. Holes should be drilled for the installation of humidity sensing elements at several locations in the structure. The relative humidity developed in small enclosed spaces within the concrete is a measure of its relative degree of saturation. Readings should be made at intervals throughout the year to determine the relationship between relative saturation and resistivity of the concrete.

3. An experimental installation consisting of a coating applied to the deck slab only as described under Method 6 should be made and the results observed at intervals by means of electrical and humidity measurements.

4. An experimental installation of cathodic protection should be made on a few units of the bridge. The term "unit" is here used to indicate a member or group of members in which the reinforcement is electrically connected.

5. Laboratory experiments, dealing with the theoretical aspects of corrosion of steel in concrete, which are now under way, should be continued.

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REFERENCE

1. "The Deterioration of Reinforced Concrete Structures under Coastal Conditions"

by S. Halstead and L.A. Woodworth,

Transactions of the South African Institution of Civil Engineers, April, 1955, pages 1-20, also discussion of above paper, October, 1955, pages 353-372

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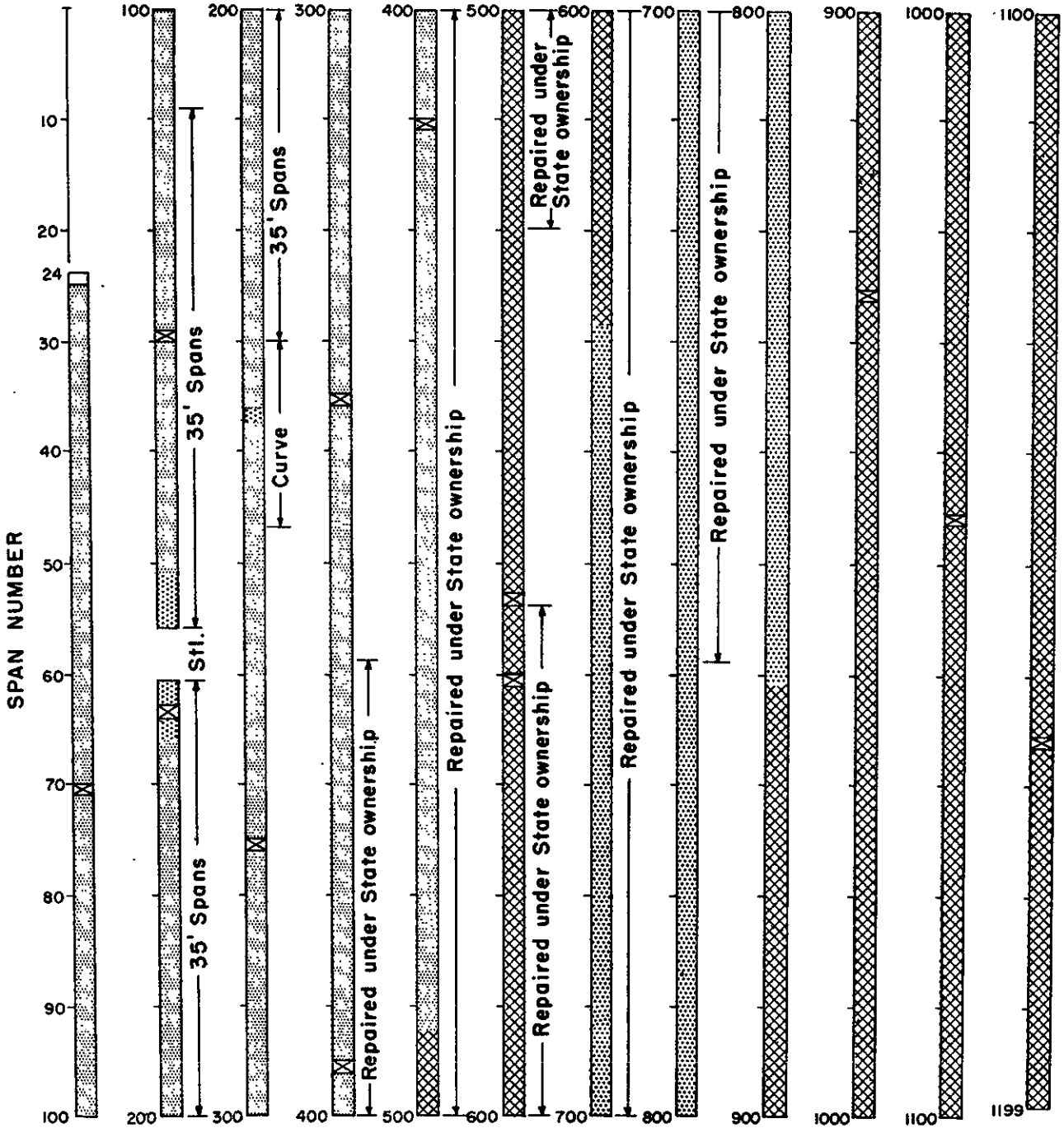

LEGEND

Test Span				
Bituminous Coating on	Bottom of deck	Beams	Caps	Piles
	✓	✓	✓	✓
		✓	✓	✓
				✓

FIGURE 1

SCHEMATIC LAYOUT OF SAN MATEO - HAYWARD BRIDGE (AS OF MAY, 1955)

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EQUIPOTENTIAL CONTOURS

SPAN 70

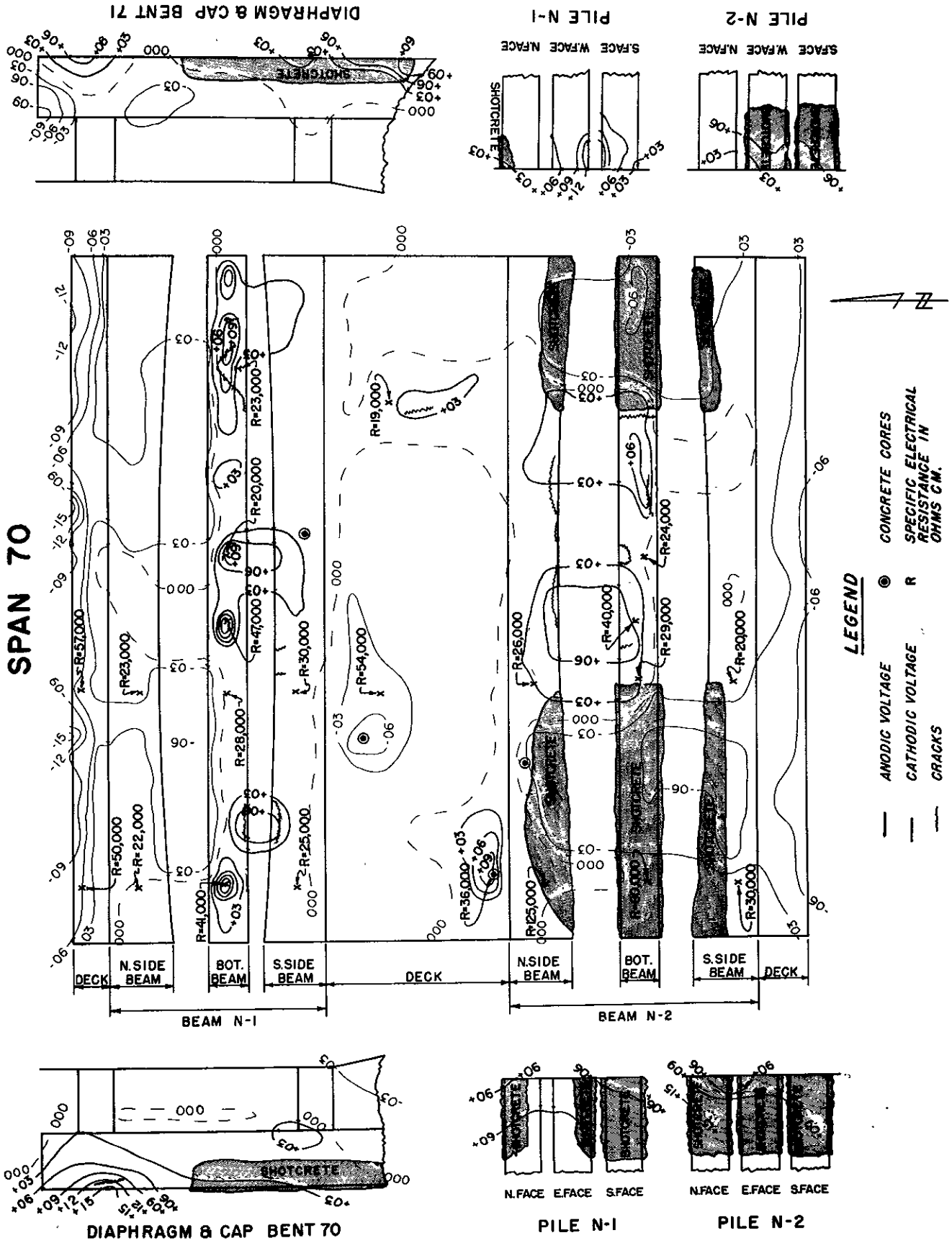

FIGURE 2

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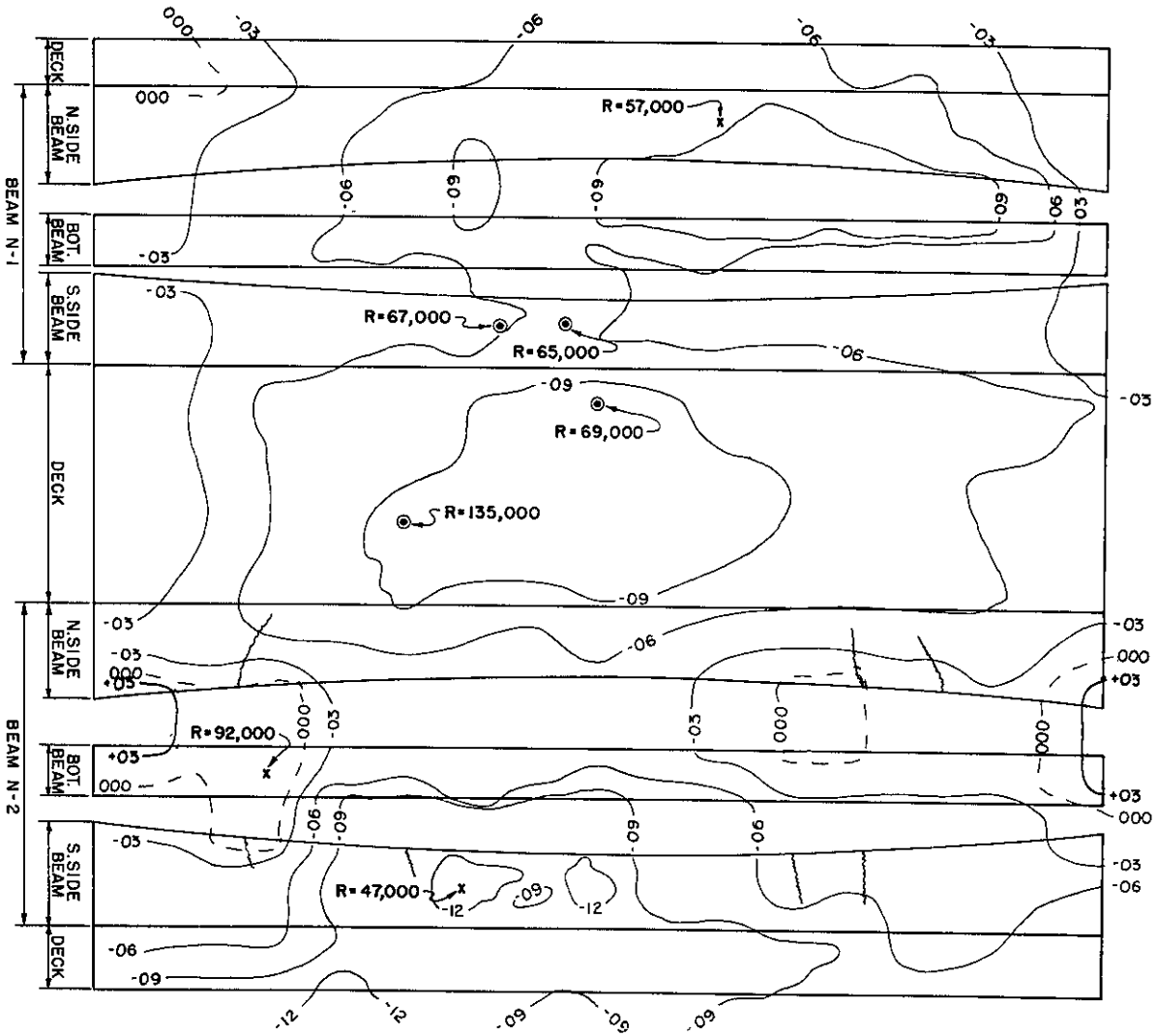
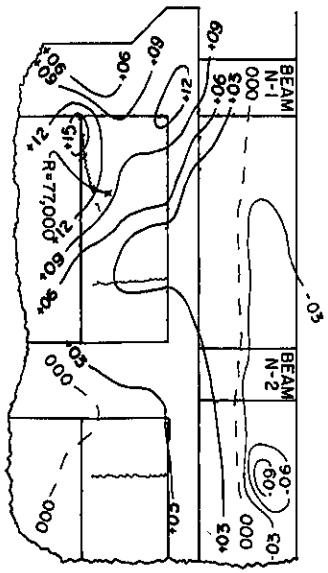
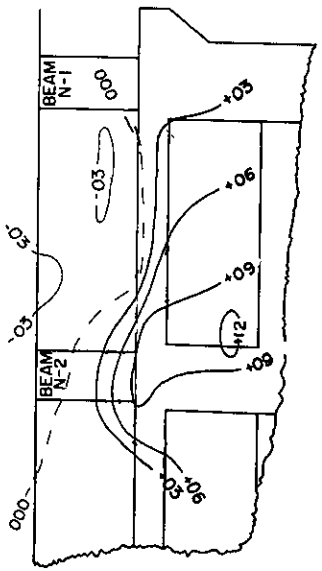
EQUIPOTENTIAL CONTOURS

SPAN 163





DIAPHRAGM & CAP BENT 163



DIAPHRAGM & CAP BENT 164

LEGEND

- ANODIC VOLTAGE
- CATHODIC VOLTAGE
- CRACKS
- CONCRETE CORES
- R — SPECIFIC ELECTRICAL RESISTANCE OF A CENTIMETER CUBED VOLUME OF CONCRETE IN OHMS

FIGURE 3

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EQUIPOTENTIAL CONTOURS

SPAN 553

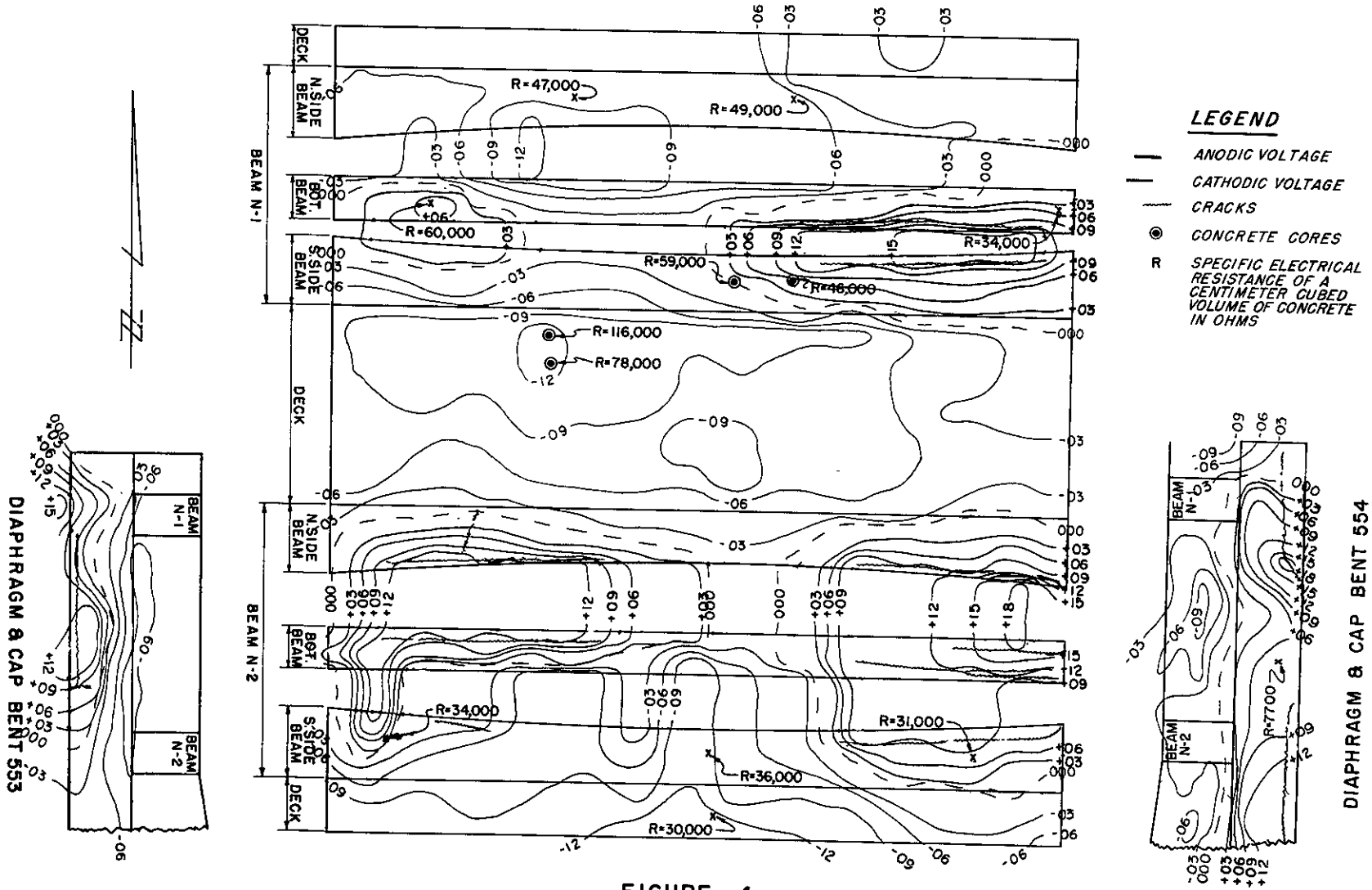

FIGURE 4

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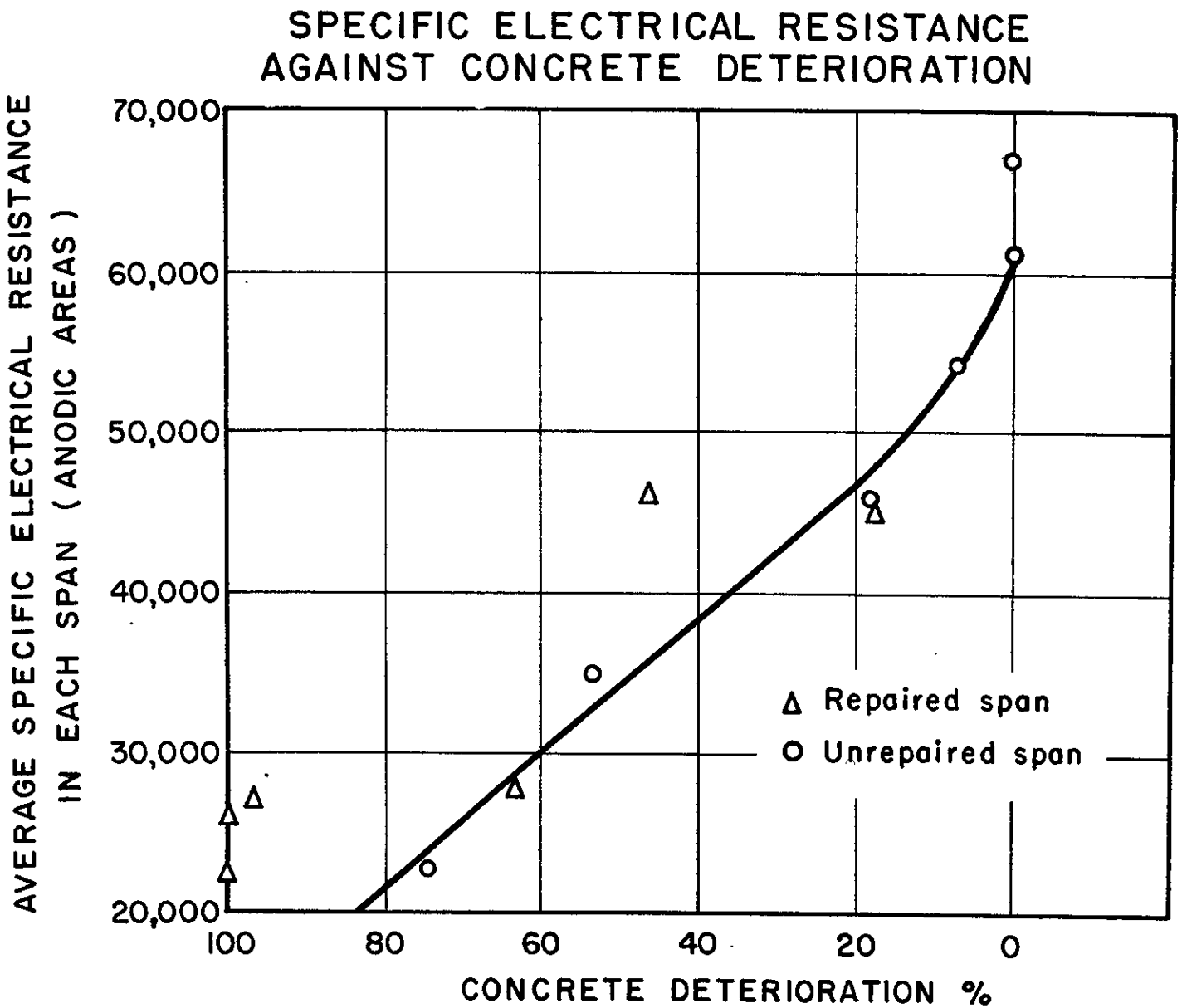

FIGURE 5

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CORROSION CELL RATIO AGAINST CONCRETE DETERIORATION

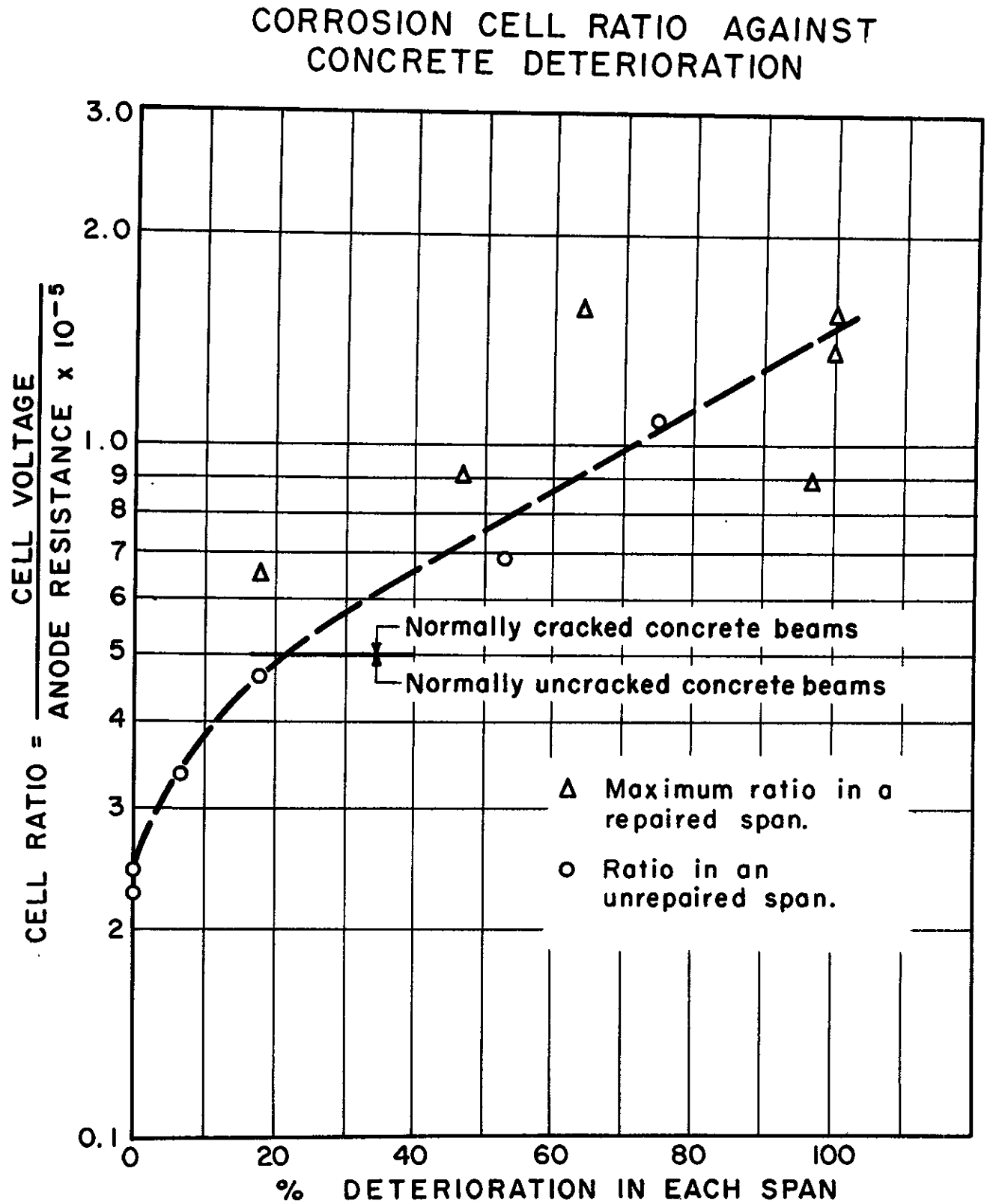

FIGURE 6

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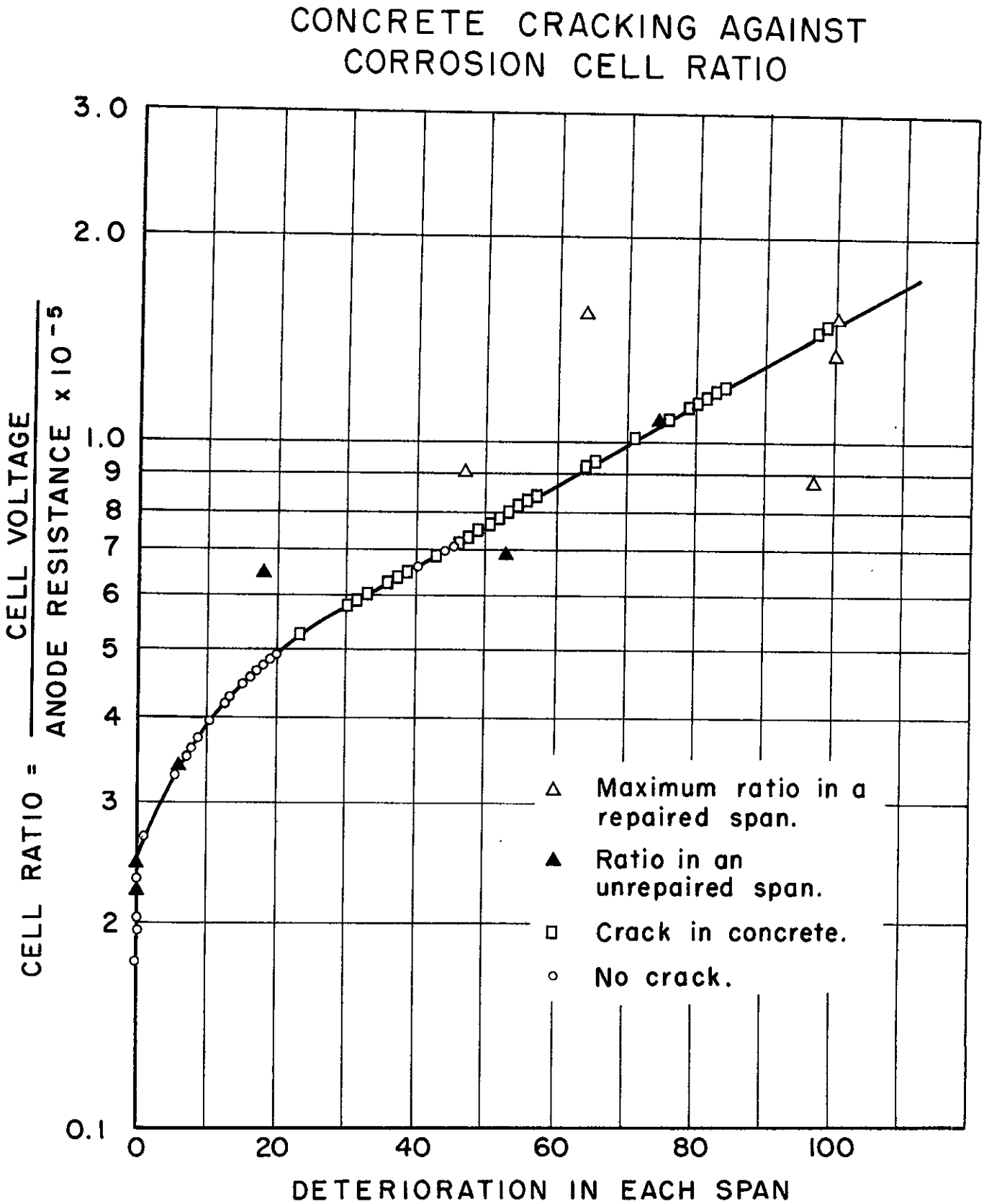

FIGURE 7

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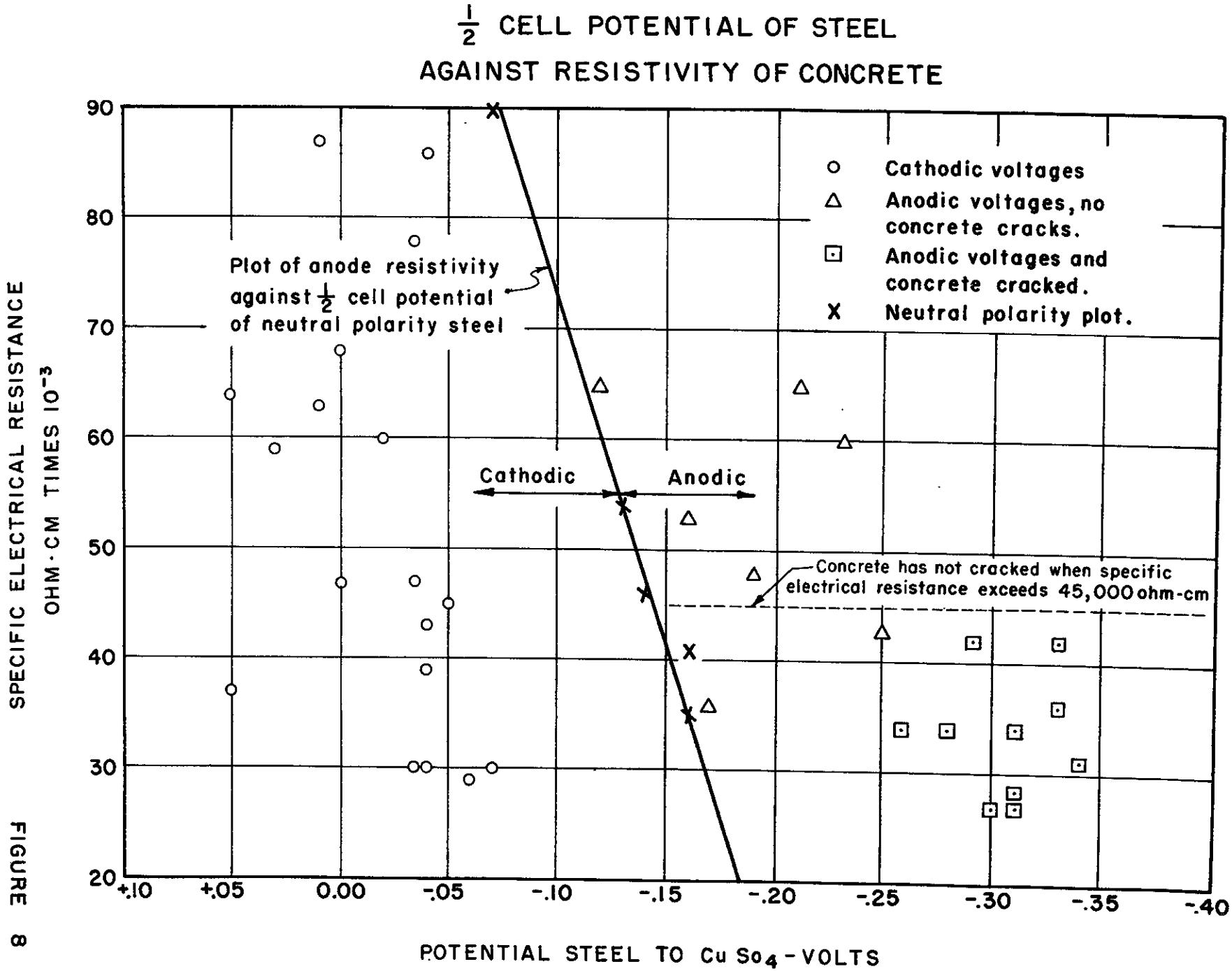

FIGURE 8

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RELATIVE HUMIDITY UNDER BRIDGE
AND SHADED FROM SUNLIGHT. JULY-OCT. 1955

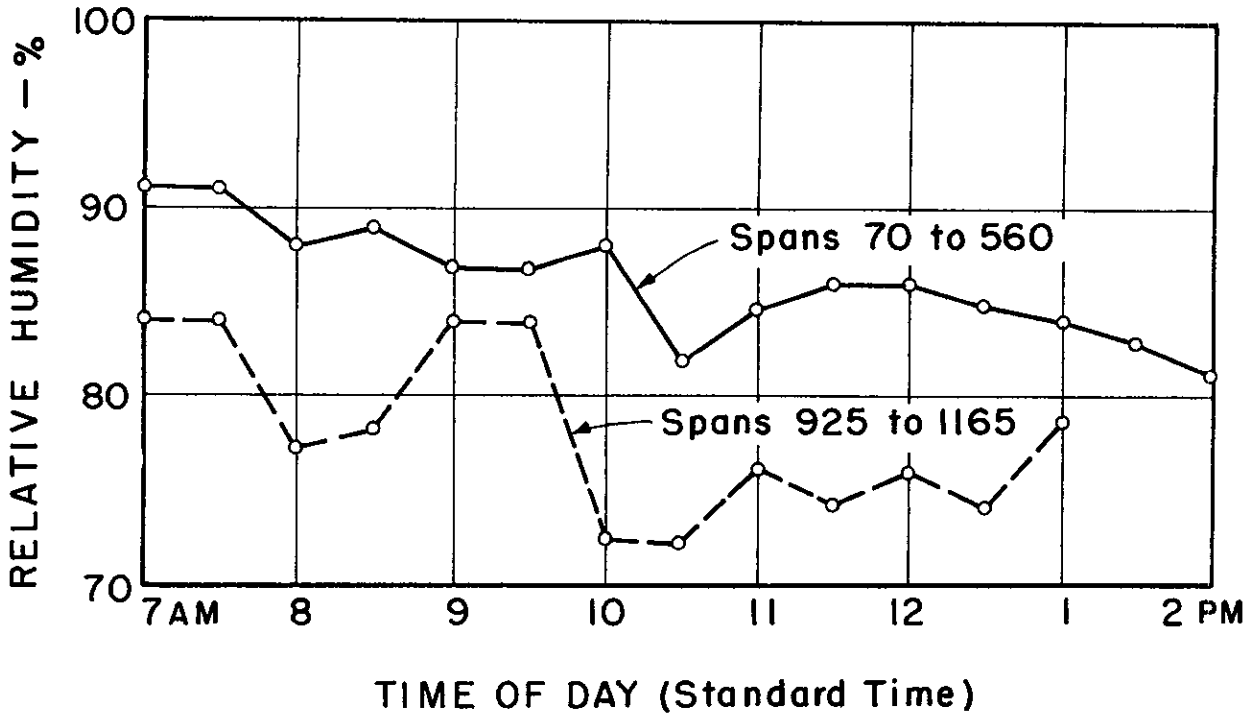

FIGURE 9

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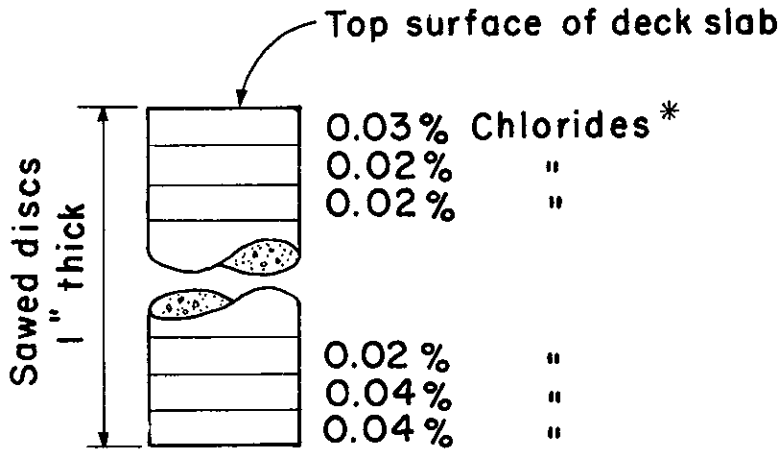
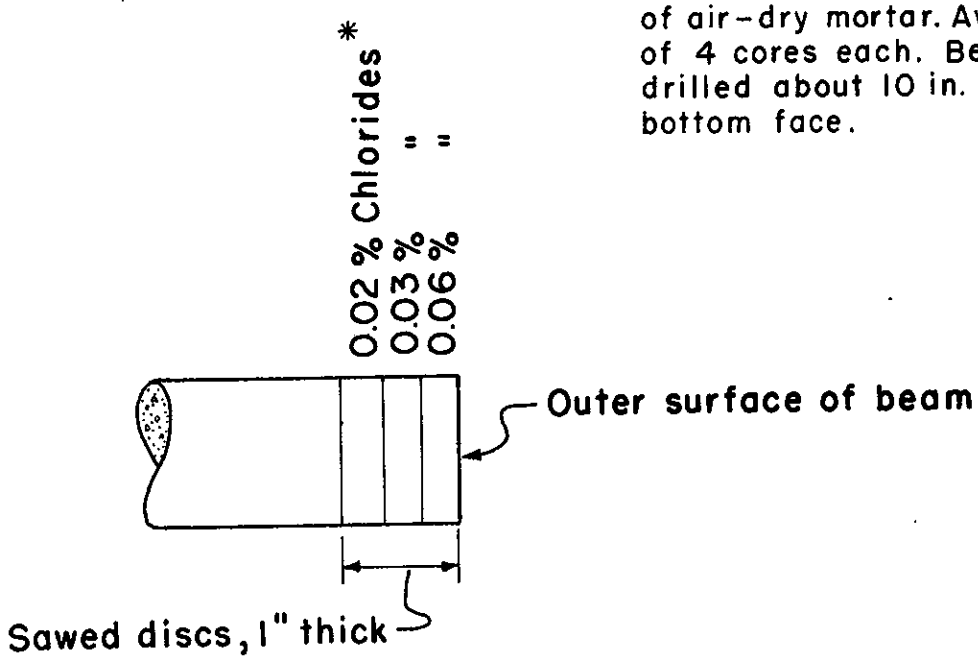
DISTRIBUTION OF CHLORIDES IN CORES FROM SPANS 70 AND 129



Cores from Deck Slabs
(drilled vertically)

* % Chlorides by weight of air-dry mortar. Average of 4 cores each. Beam cores drilled about 10 in. above bottom face.



Cores from Beams
(drilled horizontally)

FIGURE 10

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TABLE I

Physical and Chemical Properties of Cores cut from Beams
(SO_3 and MgO as percentage of original cement.
Chlorides, Cl, as percentage of air-dry mortar.)

Span No.	Core No.	Comp. Str. psi	E x 10^{-6}	Pulse Veloc. Ft/Sec	Weight lbs. per cu.ft.	Absorption %	SO_3 %	MgO %	Cl %	Resistivity ohm-cm.
70	1	3240	2.94	11,000	150.3	6.3	1.4	6.9	0.04	30,000
	2	3580	2.78	9,400	149.6	6.3	1.7	6.8	.02	26,000
129	5	3650	2.56	11,400	150.6	6.4	1.7	6.1	.04	51,000
	6	3800	2.63	12,000	152.2	6.1	1.8	7.0	.13	31,000
163	9	2770	2.38	11,800	149.6	6.4	2.2	7.8	.06	65,000
	10	3120	2.33	12,000	150.1	6.5	2.2	8.1	.06	67,000
202	13	3460	2.27	12,000	146.9	5.8	1.9	7.8	.04	24,000
	14	3200	2.33	12,800	150.3	5.6	2.2	8.3	.15	23,000
275	17	3000	2.50	12,500	149.0	6.8	2.2	8.2	.11	
	18	2960	1.96	11,400	150.2	6.5	2.0	8.4	.06	
335	21	4150	2.78	12,300	151.9	6.1	2.0	8.2	.11	25,000
	22	3760	2.38	12,400	152.2	6.5	2.0	5.9	.08	25,000
410	25	3540	2.86	13,100	149.6	6.6	2.5	6.4	.04	
	26	4110		12,400	148.5	6.8	2.5	6.6	.03	
495	29	4100		12,400	151.3	6.1	2.6	7.7	.07	45,000
	30	3560	2.78	12,000	149.6	6.5	2.1	7.7	.08	46,000
553	33	3330	2.94	11,800	150.8	6.3	2.2	5.4	.23	
	34	3300	2.78	12,100	150.1	6.7	2.2	6.3	.17	
560	37	3730	2.63	12,500	149.9	7.0	1.6	7.8	.18	77,000
	38	3380	2.70	12,600	148.7	7.0	2.3	6.8	.28	48,000
925	41	2720	3.03	12,300	151.0	6.2	2.0	8.3	.10	
	42	3710	2.86	12,400	150.8	6.0	1.8	7.8	.10	
1045	45	3920		11,200	152.2	6.6	1.7	8.1	.11	85,000
	46	4080		11,500	152.4	6.4	1.9	6.1	.11	70,000
1165	49	3860	2.56	11,900	151.5	5.7	1.6	6.7	.09	74,000
	50	4670	2.56	12,200	152.9	5.7	2.1	7.2	.08	59,000
Average		3610	2.62	12,000	150.5	6.3	2.0	7.3	0.10	48,000

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TABLE II

Physical and Chemical Properties of Cores cut from Deck Slab
(SO_3 and MgO as percentage of original cement.
Chlorides, Cl, as percentage of air-dry mortar.)

Span No.	Core No.	Comp. Str. psi	E x 10^{-6}	Pulse Veloc. Ft/Sec.	Weight lbs. per cu. ft.	Absorption %	SO_3 %	MgO %	Cl %	Resistivity ohm-cm.
70	3	3660		10,000	149.9	6.4	1.5	6.3	0.03	36,000
	4	3630		13,300	149.7	6.4	1.8	6.9	.04	54,000
129	7	3110	2.33	11,200	148.9	6.8	1.5	6.5	.01	20,000
	8	3080	2.94	11,200	148.5	6.5	1.7	6.4	.02	
163	11	3110		12,200	149.8	6.4	2.3	8.4	.03	69,000
	12	3000		12,400	148.1	6.5	2.0	8.9	.02	135,000
202	15	3590	2.94	12,400	149.0	6.9	1.9	8.3	.02	38,000
	16	3520		12,200	148.8	6.4	1.8	8.2	.03	23,000
275	19	2850	1.96	11,400	147.5	7.5	1.8	7.5	.02	
	20	2640		11,500	148.3	6.8	1.8	6.8	.03	
335	23	3200	2.44	11,800	149.1	6.8	1.8	8.0	.02	113,000
	24	3110	2.22	11,800	149.8	6.5	1.7	7.3	.01	84,000
410	27	3290		12,600	148.9	6.5	1.8	5.9	.02	71,000
	28	3570	3.13	12,400	148.6	6.7	1.5	6.2	.03	73,000
495	31	3600		12,100	148.1	7.3	1.6	5.1	.01	115,000
	32	3210	2.04	12,000	151.3	6.6	1.5	6.0	.02	102,000
553	35	3150	2.38	12,000	147.8	7.3	1.7	6.5	.02	116,000
	36	2940	1.89	11,200	148.7	6.7	1.5	6.1	.01	78,000
560	39	3240	2.33	11,900	150.7	7.1	2.0	7.2	.04	86,000
	40	3120		11,100	148.3	7.8	1.7	7.8	.02	125,000
925	43	3970	3.03	12,400	150.1	6.3	1.6	7.2	.01	43,000
	44	4020	2.23	12,400	150.8	6.1	1.5	7.2	.01	39,000
1045	47	3500	2.17	11,200	147.8	7.9	1.5	7.1	.01	45,000
	48	3560	2.08	11,200	147.6	7.7	1.5	7.2	.01	58,000
1165	51	4490		11,800	149.5	6.9	1.5	6.5	.02	120,000
	52	4400	2.50	11,800	149.7	6.2	1.3	7.0	.01	148,000
Average		3410	2.41	11,800	149.1	6.8	1.7	7.0	.02	78,000

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TABLE III

Average Pulse Velocities and Rebound Numbers of Concrete Measured in Place in the Structure

	Pulse Velocity			Rebound No.	
Span No.	Beams	Cap	Piles	Beams	Cap
70	12,800				
129	12,800				
163	12,800				
202	12,100				
275	12,200	12,000	13,600		
335	12,600	12,400	12,600		
410	13,800	13,300	13,400		
495	13,100	13,000	13,900		
553	12,700	12,800	13,800	50	43
560	12,900	13,300	12,600		
925	13,500	14,700	12,300	47	
1045	13,000	13,000	13,500	47	
1165	13,900	14,800	13,200	46	
Average	12,900	13,200	13,200		

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TABLE IV

Average Test Values in Spans Arranged in Order of Deterioration

Span No.	Percent of Cracking	BEAM CORES						Pulse Velocity in Place in Beams Ft. per Second	Resistivity in Anodic Areas ohm-cm.
		Comp. Str. psi	E x 10^{-6}	Weight lbs. per cu. ft.	Absorption %	MgO %	Chlorides %		
129	100	3720	2.60	151.4	6.2	6.5	0.08	12,800	26,000
202	95	3330	2.80	148.6	5.7	8.1	0.10	12,100	27,000
275	80	2980	2.23	149.6	6.6	8.3	0.08	12,200	
335	75	3950	2.58	152.1	6.3	7.1	0.10	12,600	23,000
	88	3490	2.55	150.4	6.2	7.5	0.09	12,400	25,000
70	60	3410	2.86	149.9	6.3	6.9	0.03	12,800	28,000
553	50	3310	2.86	150.5	6.5	5.8	0.20	12,700	35,000
560	45	3560	2.67	149.3	7.0	7.3	0.23	12,900	46,000
	52	3430	2.80	149.9	6.6	6.7	0.15	12,800	36,000
495	20	3830	2.78	150.5	6.3	7.7	0.08	13,100	45,000
925	18	3770	2.94	150.9	6.1	8.0	0.10	13,500	46,000
1045	12	4000		152.3	6.5	7.8	0.11	13,000	54,000
1163	3	4270	2.56	152.2	5.7	6.9	0.08	13,900	61,000
163	0	2950	2.35	149.8	6.4	8.1	0.06	12,800	67,000
	10	3760	2.66	151.1	6.2	7.7	0.08	13,300	55,000

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