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RELATIONSHIP BETWEEN PROPERTIES OF HARDENED CONCRETE AND BRIDGE DECK PERFORMANCE IN VIRGINIA

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(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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ABSTRACT

Among Virginia's research efforts during the 1960's was the study of concrete placed in 17 randomly selected bridge decks during and subsequent to their construction in 1963, with the purpose of relating the properties of the concrete as placed to its subsequent performance. The project reported here examined these decks and concrete samples removed from them after 14 years of service, again to relate performance to properties and, based upon this relationship, to suggest initial levels of concrete properties required for good performance, particularly where concrete as placed does not comply with specific requirements.

Despite the relatively small number of decks and samples, as viewed in comparison to the variables involved, significant relationships among concrete properties and performance are evident from the data, in that resistance to scaling and chloride penetration has been maintained for 14 years by concrete that met the requirements established by American Concrete Institute Committee 201 on Durability. The importance of long-established principles for producing durable concrete -- i.e., low water-cement ratio, consolidation, curing, and cover depths -- thus have been reconfirmed.

A procedure for evaluating the potential durability of concrete using petrographic examinations combined with estimates of service and environmental conditions was developed, and the preliminary application of this procedure to the decks included in this study were encouraging.

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INTRODUCTION AND BACKGROUND

Coinciding with the nationwide concern for the premature deterioration of concrete bridge decks that developed in the 1960's, were the Virginia Department of Highways' initiation of research and upgrading of its specifications in an effort to improve the performance of its bridge decks. Studies of the "bridge deck problem" have been extensively reported. (National Cooperative Highway Research Program 1970 and 1979).

Virginia's research involved two major efforts, one of which included a performance survey of a comparatively large sample of decks randomly selected to represent all of the decks in the highway system. The purpose of this survey, made in 1960 and in 1969 and reported by the U. S. Bureau of Public Roads et al. (1969), was to identify the types of deterioration and their frequency of occurrence. The second research effort involved intensive study of the concrete in a small number of decks during and subsequent to construction, and the purpose was to relate the properties of the concrete to the defects observed (Newlon 1971).

The 1960 survey was made as part of a nationwide project in which Virginia was one of eight states in which the performance of bridge decks in service was studied. In a follow-up study conducted by the Research Council ten years later, these same decks were again surveyed (Davis, North, and Newlon 1971).

These surveys showed that some forms of deterioration of concrete, particularly deicer scaling, constituted a major problem in Virginia, while spalling from corrosion of the reinforcing steel, a major problem in some states, was found to occur infrequently. Thus, the primary defects to which the Department's effort was initially directed were those involving deterioration of the concrete itself rather than problems resulting from corrosion of the reinforcement, which has subsequently received major attention nationwide.

The second major research effort in Virginia, that reported by Newlon in 1971, was an intensive study of the concrete on 17 randomly selected bridges constructed during 1963-64. Included were bridges whose decks had been screeded with the different types of mechanical equipment then coming into use as hand screeding and finishing were being phased out of construction operations. This project documented that the major problems at the time were the borderline quality of the concrete being used and a lack of application of concreting procedures for placing, finishing, and curing that are necessary for obtaining adequate durability under the increased traffic volumes and loads associated with the interstate system. Major deficiencies included higher water-cement ratios and lower air contents than needed. In general, the performance after seven years reflected and correlated with these deficiencies (Newlon 1971).

As a result of these studies, specifications were upgraded, training and certification programs were instituted, and increased attention was directed toward evaluating new techniques and materials and improving the use of traditional ones. The improved performance of concrete in bridge decks that resulted from these several efforts was noted in a subsequent statewide performance survey (Newlon 1974).

Even though concrete currently specified is of improved quality and performance, occasionally concrete not fully complying with the specifications and of borderline quality is unintentionally supplied. When identified after placement, a decision must be made as to the degree to which performance will be detrimentally affected if the concrete not meeting specifications is left in place. Whether or not quality assurance is based upon traditional or statistical concepts, the decision must be made to leave the concrete in place, remove it, or institute a remedial procedure. For these required judgments, some documentation of the field performance of "borderline concrete" would be helpful. The decks included in the 1963-64 research project were seen to provide an excellent opportunity to gain the needed information on performance, since extensive data taken on the properties of the freshly mixed and hardened concrete prior to opening the decks to traffic were available (Newlon 1971). Additionally, it was anticipated that study of these decks should suggest relationships between properties and performance with respect to deteriorative processes such as corrosion of reinforcement for which concern has increased since the 1971 report was issued.

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OBJECTIVES

The purpose of the research reported in this report was twofold:

1. To develop relationships between the performance of concrete in bridge decks under service conditions and the initial properties of the concrete, some of which was of "borderline" quality.

2. To suggest, based upon performance, initial levels of concrete properties that should be demanded before a decision is made to remove or apply remedial measures to a deck when the concrete as placed does not meet specifications.

STUDY SAMPLE

Characteristics of Bridges

Important characteristics of the test bridges are included in Table 1, wherein the bridges are grouped according to the screeding method used, a major variable of the original study (Newlon 1971). Plans, geometric details, location of test samples, and other information are included in Figures A-1 through A-17 taken from the final report on the study (Newlon 1971). For each of the bridges, one deck was selected for testing and observation. On each of the selected decks, two batches of concrete were randomly selected during construction for determinations of the properties of the freshly mixed concrete. The locations of these batches in the decks were carefully established, and for petrographic examination a core was removed from each location prior to opening the deck to traffic. Periodic performance surveys have been made with special attention being given to the sample locations. The condition surveys have employed the procedures and reporting format developed in the original project of the U.S. Bureau of Public Roads et al. (1969), as will be discussed later. A typical survey sheet is shown in Figure A-18.

The 17 bridges reflect a wide range of construction materials and traffic conditions. In 1977, at the time of the final survey of the 17 bridges, 5 (29%) had been overlaid before the survey for various reasons: one bridge (#5) was subsequently overlaid, but is treated as uncovered in this report. Because of the comparatively limited number of structures as compared with the number of variables involved, definitive relationships between performance and properties would not be expected, but consistent relationships that are identified would be expected to apply to a wide variety of decks. The various factors affecting performance will be discussed at appropriate places in the report.

Table 1

Important Characteristics of Test Bridges

Job No.	Screeding equipment	Materials Coarse aggregate	Materials Fine aggregate	Water-reducing retarders	Curing Method	1969 traffic count, VPD	Structure type*
1	Vibrating	Natural siliceous gravel	Natural siliceous gravel	Yes	Paper	24,010	PS-IB-SN
12	Vibrating	Natural siliceous gravel	Natural siliceous gravel	No	White pigmented compound	48,435**	SS-IBWG-SC
2	Mechanical oscillating (transverse)	crushed limestone	crushed limestone	No	White pigmented compound	8,600	PS-IB-SC
5	Mechanical oscillating (transverse)	Crushed limestone	Crushed limestone	No	White pigmented compound	12,851	SS-IB-SC
15	Mechanical oscillating (transverse)	Natural siliceous gravel	Natural siliceous gravel	Yes	White pigmented compound	31,565	SS-IB, DG-SC
16	Mechanical oscillating (transverse)	crushed limestone	crushed limestone	Yes	White pigmented compound	25,935	SS-IB-SC
3	Mechanical oscillating (longitudinal)	Crushed granite	Natural siliceous sand	No	White pigmented compound	354	PS-IB-SN
4	Mechanical oscillating (longitudinal)	Crushed limestone	Natural siliceous sand	No	White pigmented compound	1,575	SS-IB-SC
9	Mechanical oscillating (longitudinal)	Crushed granite	Natural siliceous sand	No	White pigmented compound	6,725	PS-IB-SN
14	Mechanical oscillating (longitudinal)	Crushed limestone	Natural siliceous sand	No	White Pigmented compound	13,375	SS-IB-SC
6	Hand	Natural siliceous gravel	Natural siliceous sand	No	Wet burlap plus polyethylene	25,925	SS-IB-SC
7	Hand	Natural siliceous gravel	Natural siliceous sand	Yes	Polyethylene	25,925	SS-DG-S
8	Hand	Crushed granite	Natural siliceous sand	No	Wet burlap	351	SS-IB-SC
10	Hand	Crushed limestone	Natural siliceous sand	No	Wet limestone dust	222	RC-SS-C
11	Hand	Crushed sandstone	Natural siliceous sand	No	White pigmented compound	445	SS-IB-SC
13	Hand	Natural siliceous gravel	Natural siliceous sand	Yes	Paper	20,345	SS-IB-SN
17	Hand	Crushed sandstone	Natural siliceous sand	No	Paper	104	SS-IB-SC

* For explanation of symbols, see Appendix B.

**The test area of the span has not received any traffic but the adjacent one-half span has been open to traffic for seven years.

Taken from Newlon (1971)

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Materials

The requirements for concrete used in bridge decks during the period 1938-1970 are shown in Table 2. All of the concrete on the projects studied was supplied from ready-mix trucks under the requirements designated "1958" in this table. The values determined during construction were reported in the 1971 report by Newlon. It will be seen that the requirements in force during the period that the bridges were built were representative of those covering a long period of construction. In important respects, notably the levels required for air entrainment and the water-cement ratio, the concrete properties specified were not adequate for high volume traffic and extensive use of deicing chemicals. As noted in the earlier report, 6% of the samples exceeded the specified slump, 22% of the reported water-cement ratios exceeded the maximum value specified, and 18% of the air content values were outside the required limits (15% below and 3% above). It is of interest to note that had the specification limits adopted in 1966 been in effect at the time of construction, 68% of the air content values would have been below the minimum and 50% of the reported water-cement ratios above the maximum values. In addition, for 41% of the samples the time between final texturing and application of curing exceeded one hour. Substantial upgrading of the specifications occurred in 1966 as a result of several interrelated factors, including input from the several research projects (U.S.BPR et al. 1969; Newlon 1971) with concomitant improvement in performance (Newlon 1974).

Traffic Volumes

When surveyed in 1977, the decks were subjected to a wide range of traffic as was indicated in Table 1. The traffic volumes would, in general, reflect the tendency to wear, the frequency of deicer application, and load related distress. The traffic data will be used later in the report.

Environmental Factors and Deicing Policy

In an earlier report (Davis, North, and Newlon, 1971), the environmental factors of importance to bridge deck performance in Virginia were summarized based upon climatological data from a variety of sources. These factors vary widely throughout the state and attempts to characterize them provide only an estimate at best. All of the bridges studied lie within the weathering region designated as "severe" in ASTM Designation C-62 "Specifications for Clay Building Brick", which is also used in ASTM Designation C-33 "Specifications for Concrete Aggregates." Using the data from the 1971 report the exposure conditions for the 17 decks are indicated in Table 3. The bridges are shown in decreasing order of estimated deicer applications. The estimated 1978 traffic data are also shown in Table 3.

Table 2

Requirements for Bridge Deck Concrete 1938-1970

	Cement Content,* lbs/yd³	(kg/m²)	Water-Cement Ratio,** by wt.	Air Content, %	Slump Inches (cm)	Max. Agg. Size, Inches	(mm)	28-day Strength, psi (MPa)	L.A. Abrasion Loss % Coarse Aggregate 100 rev.	500 rev.	Sulfate Soundness loss, %*** coarse aggregate / fine aggregate	F & T ***
1938	588	(349)	0.53	------	2-5 (5-13)	1	(25)	3000 (20.7)	10	40		10(15)
1947	588	(349)	0.53	----****	2-5 (5-13)	1	(25)	3000 (20.7)	9	35	8(5) / 8(5)	5(15) / 5(15)
1954	588	(349)	0.53	3-6****	0-5 (0-13)	1	(25)	3000 (20.7)	9	35	8(5) / 8(5)	5(15) / 5(15)
1958	588	(349)	0.49	3-6	0-5 (0-13)	1	(25)	3000 (20.7)	9	35	8(5) / 8(5)	5(15) / 8(15)
1966	634	(376)	0.47	$6\frac{1}{2}\pm1\frac{1}{2}$	2-4 (5-10)	1	(25)	4000 (27.6)	9	40	12(5) / 12(5)	5(20) / 5(20)
1970	682	(405)	0.47	$6\frac{1}{2}\pm\frac{1}{2}$	2-4 (5-10)	1	(25)	4000 (27.6)	9	40	12(5) / 18(5)	5(20) / 8(20)

* The correct contents correspond to those as conventionally expressed as follows: 588 lb/yd^3 = $6\frac{1}{4}$ sk/yd^3; 634 lb/yd^3 = $6\frac{3}{4}$ sk/yd^3; 682 lb/yd^3 = $7\frac{1}{4}$ sk/yd^3

** The water-cement ratios correspond to those as conventionally expressed as follows: 0.53=6 gal/sk 0.49=$5\frac{1}{2}$ gal/sk 0.47=$5\frac{1}{4}$ gal/sk

*** Values in parentheses are specified number of cycles.

**** Air entrainment was used in pavements beginning in 1948. It was used experimentally in several bridge decks prior to incorporation into specifications.

Taken from Newlon (1974)

Table 3

Exposure Conditions

Bridge Number	Estimated Total Deicer Applications	Age When Opened to Traffic, Months	Estimated 1978 Daily Traffic	Estimated Cumulative Effective Freeze & Thaw Cycles
2	780	30	9,500	480
1	702	14	18,000	325
12	700[a]	3	20,000	560
15	660	27	11,500	180
13	650	8	5,000	520
6	468	18	19,000	325
7	468	18	20,000	325
3	360	31	1,200[b]	180
9	360	25	5,600	180
4	210	6	3,500	560
16	210	3	27,000	560
5	195	12	10,000	560
14	180	16	7,000	480
8	168	5	540	280
11	140	2	270	560
10	39	9	160	585
17	28	17	280	560

a Adjacent lane

b Predominately trucks entering and leaving a truck stop

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There is a general correspondence between traffic volumes and deicer applications in that the lowest traffic volumes correspond with the least frequent deicing. There is no clear relationship between applications of deicers and potential for freezing and thawing, since deicing is more controlled by traffic volumes than by climate.

The data suggest that with the exception of #10 and #17 the bridges were exposed to considerable amounts of chloride. In addition, the portion of bridge #12 that was studied had no chlorides applied to it, but the adjacent area was heavily treated.

From the outset of the project, it was recognized that determining the amount of deicing chemicals applied to the bridges would be extremely difficult, because the amount depends upon traffic volumes and local policies as well as weather conditions. For example, several of the decks, while part of the primary or secondary systems, serve as important connections to the interstate roads or urban areas, and thus require more deicing chemicals than would be indicated based upon their classification alone. It was decided that the best estimate would be the "average number of deicing applications per year for each bridge" obtained from the field personnel in whose jurisdiction the bridge is located. The estimates so established are the basis of the information in Table 3.

During snow removal operations, chlorides were applied directly in the form of sodium chloride (NaCl) or calcium chloride ($CaCl_2$), mixtures of the two compounds, or in abrasive mixtures of sand and one or both of the chemical compounds.

Based upon their molecular weights, $CaCl_2$ and NaCl supply 0.63 and 0.60 lb (0.286 and 0.272 kg) of chloride ion per pound (Cl^-/lb[Cl 7kg]) of the compound. Thus, in considering the contribution of the deicing chemicals to chloride concentration in the concrete, the two compounds can be taken as being equivalent. When NaCl or $CaCl_2$ is mixed with, and thus diluted by, abrasives, the amount of Cl^- obviously depends upon the dilution factor. The ratio of deicing compound to the total weight of the abrasive and the compound generally ranges between 33% and 50%. This would mean that an application of abrasive mixture would supply (0.20 to 0.30 lb/Cl^- [0.09 to 0.14 g/g]/lb/mixture). Even the diluted mixture would probably supply sufficient Cl^- to be absorbed into the concrete on about the same basis as the undiluted deicing chemicals, since all conditions would give a saturated solution of chlorides.

In addition to the penetration of chlorides which influence corrosion of the reinforcement, the important influence of chlorides on accelerated deterioration by freezing and thawing -- i.e., scaling -- is well accepted. Maximum scaling occurs when the concentration of NaCl or

$CaCl_2$ in water is from 2% to 4% during freezing and thawing (Verbeck and Klieger 1957; Newlon 1978). While the concentration can obviously vary widely under field conditions, there would certainly be conditions under which the worst conditions with regard to scaling could prevail. Sufficient Cl^- would be available to cause about the same degree of scaling whether the chemicals were used with or without abrasives. It thus appears that a reasonable estimate can be made of the exposure to chlorides by considering that applications are equivalent in Cl^- whether or not the chemicals are mixed with abrasives. Using this approach the total number of chloride applications for each deck was estimated and is shown in Table 3.

The estimated exposures to freezing and thawing were derived using data developed by Russell (1943) and discussed in the report by Davis, North, and Newlon (1971). Russell considered a drop in air temperature below 28°F(-2.2 °C) followed by a rise above 32° F(0°C) as an "effective" freezing and thawing cycle. The average effective cycles multiplied by the number of years of service for each bridge is given in Table 3. It should be emphasized that these data are not intended to be definitive, but only to suggest the levels and variations of exposure. It is interesting, however, to note that over 12 to 15 years many of the decks were subjected to more cycles than are obtained in conventional laboratory testing.

PROCEDURES

Field Performance

Each of the test spans was surveyed in 1966, 1970, and 1977 using the procedure initially developed for the 1969 BPR et al. survey. Surveys of this type, while not detailed, have proved useful for assessing the general level of performance and permit comparisons of changes with time. As noted previously the survey form used with accompanying instructions is given in the Appendix. As part of the 1977 survey, measurements of electrical corrosion potentals were made using ASTM method C876 "Half Cell Potentials of Reinforcing Steel in Concrete" and a 5-ft (1.5m) grid was used. These measurements were not made in 1966 and 1970.

Laboratory Analyses

As noted earlier, one 4 in (100 mm) diameter core was removed from each sample location in 1964 prior to opening of the deck to traffic. In 1977, two 4 in (100 mm) diameter cores were removed as near to the original core locations as possible for tests of chloride content and absorption and for petrographic examination.

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Each of the cores was cut as shown in Figure 1. The shaded portions were prepared and tested for chloride content using the Berman procedures (Clear and Harrigan 1977) as modified by Clemena, Reynolds, and McCormick 1976, and the intervening portions were used for determining the absorption of the concrete at the various levels and for petrographic examination intended to provide additional information on relationships between properties and performance. Samples from corresponding levels of the companion cores were used for fabrication of thin sections used in petrographic studies.

The absorption was determined by weighing the cores after exposing them to laboratory conditions (40% to 60% relative humidity and temperatures of 72 $\pm$ 2° F(22 $\pm$ 1.1°C) for at least six months, immersing them in water, and weighing them periodically until a constant weight was achieved. The cores were then oven dried to constant weight at 212°F (100°C). The weight after immersion compared with the final oven dry weight was used to determine absorption. The examinations were conducted using procedures described later in the report.

RESULTS

A major objective of this project was to discover correlations between the field performance of the decks and observed properties of the concrete, environmental factors, and service conditions. However, since each of the 17 bridges studied was a unique case because of variations in materials, contractors, weather conditions during construction, design features and service conditions subsequent to construction, it was recognized that isolating the effects of a given variable with a high degree of assurance would be very difficult if not impossible. In addition to the large number of construction and environmental variables, the borderline properties of the concrete, which taken alone would be expected to provide a wide range of performance characteristics, would at the same time magnify the influence of variations in service conditions based upon work by Cordon and Merrill (1963). As shown in Figure 2, durability varies over a wide range when air contents are in the "transition zone." Such air contents are typical in the decks. In addition to the effect of air contents relationships between the water-cement ratio and durability have been demonstrated in studies in which an increase in the water-cement ratio from 0.45 to 0.55 reduced resistance to freezing and thawing by 50% (U.S. Bureau of Reclamation 1955). Thus, at best, the correlations between performance and individual properties will be in the form of trends and comparisons in relation to documented relationships.

Despite the limitations enumerated above, however, the performance will be shown to be explained in very large measure by factors well established in concrete technology.

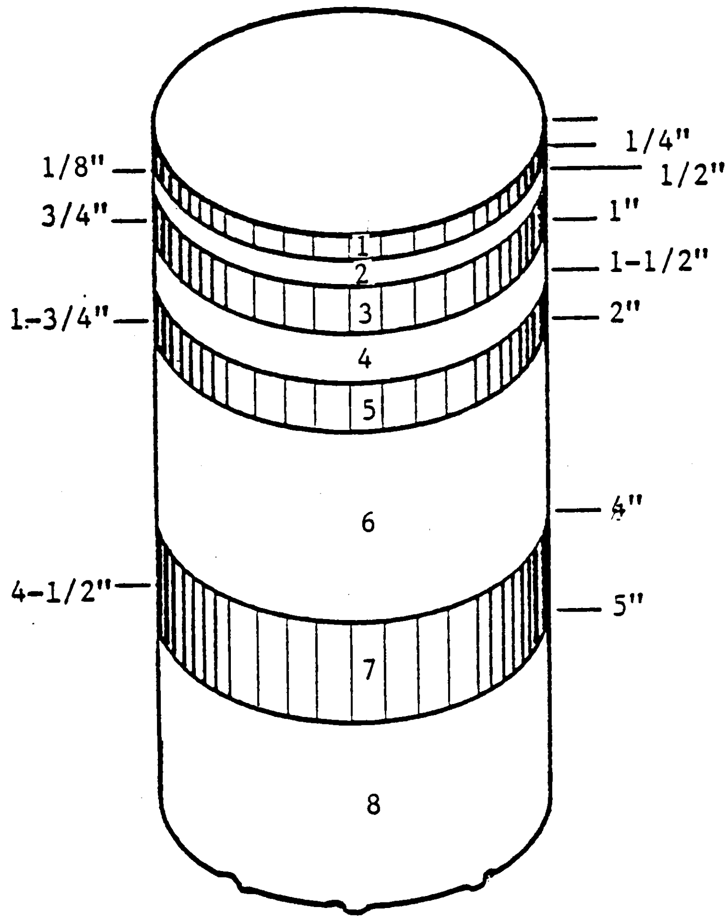

Figure 1. Location of core portions studied.

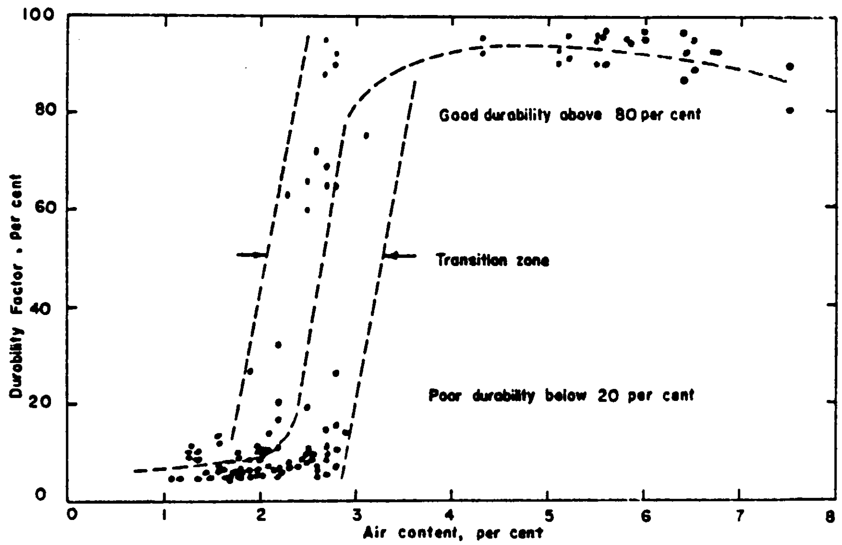

Figure 2. Durability of concrete containing various aggregates, cement contents, water-cement ratios, and air contents. (After Cordon and Merrill 1963.)

General Field Performance

Of the 17 decks, 5 had bee overlaid at the time of the 1977 survey. Of these 5, bridges #4, #9, and #14 were overlaid comparatively soon after construction, primarily as a result of insufficient cover over the reinforcement associated with the screeding method, even though other defects existed (Hilton 1971). Bridge #2 was overlaid to correct insufficient skid resistance resulting from use of a limestone fine aggregate in the concrete exposed to a high volume of traffic. Bridge #4 was overlaid after 6 years, primarily to remedy rusting of isolated bent up reinforcement without sufficient cover. In the case of bridge #14, a 2 in (50 mm) concrete overlay was placed prior to opening the deck to traffic. In the case of bridge #9, the span carried traffic for 4 years, at which time extensive patching was done prior to overlaying in 1969 and again in 1972; a major deck restoration was completed in 1977. The last of these 5 bridges, #11, was overlaid after the bridge had been in service for 12 years to correct surface defects, essentially wear and deterioration of the surface mortar. The performance of these overlaid decks is discussed later in this report.

Bridge #5 was overlaid after the 1977 performance survey was made, and is treated as uncovered in this report. The remaining 11 bridges remain uncovered.

The data from the performance surveys in 1966, 1970, and 1977 are summarized in Table 4. Because there are comparatively few bridges and many variables, definite relationships would not be expected. The data, however, illustrate tendencies in performance that can be related to observed properties of the concrete or construction practices. It should be borne in mind that the ability to observe cracking characterized as "light" depends strongly on the environmental conditions at the time of inspection. In one or two cases light cracking that had been indicated was not observed in a subsequent inspection. It is also possible that a defect identified as light pattern cracking progressed to surface scaling. In some cases it is difficult to distinguish between surface scaling and wear. Where the difference was obvious, the distinction is indicated. Considering that the surveys were made by different people at different times without reference to prior survey results, the consistency is encouraging.

In general, the performance would be characterized as borderline, and reflects the borderline quality of the concrete and construction practices. The most prevalent defect was deterioration of the surface mortar or scaling caused by deicers. All but two of the structures showed this defect. Four of the decks had transverse cracking classified as medium or heavy. The remaining defects were comparatively infrequent. Specific defects are discussed later in the report.

Bridges #16, #10, and #11 are of interest as extreme examples. The performance of bridge #16 was exceptionally good considering the traffic and deicing chemicals to which it had been exposed. It was essentially free of defects. At the other extreme was the relatively poor performance of bridges #10 and #11 in view of the comparatively light traffic and few applications of deicing chemicals to which they had been subjected.

Table 4

Field Performance Based Upon Visual Survey

	Scaling, Percent			Cracking, Predominance & Severity															Rusting			Surface Spalls, No.			Joint Spalls, No.			Popouts			1977 Avg. Veh. Day	Est. No. Chloride App.
				Transverse			Longit.			Diagonal			Pattern			Random																
No.	66	70	77	66	70	77	66	70	77	66	70	77	66	70	77	66	70	77	66	70	77	66	70	77	66	70	77	66	70	77		
1	5L	30L	80L	0	L	M	L	M	0	0	0	M	0	0	L	L	0	L	0	0	0	0	0	1S	0	0	0	0	0	0	18,000	702
2	0	0	X	0	L	X	0	0	X	0	0	X	0	0	X	0	0	X	0	0	X	0	0	X	0	0	X	0	0	X	9,500	780
3	0	40L	40M	0	0	L	0	0	0	0	0	0	0	0	L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	X	1,200[1]	360
4	5L	15L	X	0	L	X	0	L	X	0	0	X	L	L	X	0	L	X	0	0	0	0	10_s[2]	X	0	0	X	0	0	X	3,500	360
5	10L	15L[3]	70M[4] 60M	0	L	L	0	L	L	0	0	0	L	L	L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	10,000	560
6	20L	35L[5]	25H 15S	0	0	M	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1_s	0	0	1_s	0	0	f	f	19,000	468
7	0	30L	80M 10H	L	0	M	0	0	L	0	0	0	0	L	M	0	0	M	0	0	0	0	0	3_s	0	0	0	0	0	0	20,000	463
8	0	15L	15L	0	0	0	0	0	0	0	0	0	0	0	0	0	L	0	0	0	0	0	0	1L	0	0	0	0	0	0	540	168
9	10L	X	X 15L	L	X	X	0	X	X	L	X	X	0	X	X	0	X	X	0	X	X	0	X	X	0	X	X	0	X	X	5,600	360
10	20L	50L[3]	85M	0	0	0	0	0	0	0	0	L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	160	39
11		50L[6]	X 30L	0	0	X	0	0	X	0	0	X	L	L	X	0	0	X	0	0	X	0	0	X	0	0	X	0	0	X	270	140
12	0	20L	30M 40L	0	0	0	0	0	0	0	0	0	L	L	0	0	L	0	0	0	0	0	0	0 5_s	0	0	0	f	f	m	20,000*	700*
13	0	15L	10H	0	L	H	0	L	M	0	0	L	0	0	L	0	L	M	0	0	0	0	0	1L	0	0	0	f	f	m	5,000	650
14	X	X	X 10L	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	7,000	180
15	0	20L	15M	L	L	L	0	0	0	0	0	0	0	0	L	0	L	L	0	0	0	0	0	10_s	0	0	10	0	0	f	10,000	195
16	0	0	0 25L	0	L	0	0	L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	27,000	210
17	0	0	10M	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	280	28

NOTES: 1 - Predominantly trucks entering and leaving truck stop, 2 - Over bent up bars without cover, 3 - Predominantly wear, 4 - On west 2/3 of span, 5 - Some Heavy, 6 - Some Medium

LEGEND: X - Overlaid, L - Light, M - Medium, H - Heavy, S - Severe, f - few, m - many

*Adjacent lanes

Chloride Related Performance

Chlorides used as deicers influence the performance of bridge decks in two ways: (1) scaling, i.e., deterioration of surface mortar from freezing and thawing, may be greatly accelerated, and (2) corrosion of the reinforcement in the deck may be caused by salt intrusion through the concrete. Good quality concrete can mitigate both of these effects of exposure to chlorides. With regard to scaling, The American Concrete Institute, in its Guide to Durable Concrete (ACI 1977), states that concrete exposed to a combination of moisture, cyclic freezing, and ice removal agents requires the following:

1. Design of the structure to minimize exposure to moisture
2. Low water-cement ratio
3. Air entrainment
4. Suitable materials
5. Adequate curing
6. Special attention to construction practices

Resistance to corrosion is obviously a function of the ability of chloride ions to reach the reinforcing steel, and thus depends upon the permeability and amount of concrete cover over the reinforcement. Permeability of concrete depends largely upon the water-cement ratio and the degree of consolidation and curing of the concrete, which are affected by construction operations such as finishing and texturing.

The field performance of the concrete in the decks will be discussed in relationship to the requirements identified by the ACI. It should be emphasized, however, that the requirements in force at the time of construction, as shown in Table 2, were subsequently recognized as being deficient and have since been upgraded to conform to those suggested by the ACI.

Scaling

The primary factors providing resistance to deicer scaling are (1) a proper air void system, (2) low water-cement ratio, and (3) adequate curing. The generally accepted requirements for an adequate air void system are (1) about 9% air in the mortar fraction; (2) a number of voids per linear inch that is significantly greater than the numerical value of the percentage of air in the concrete; (3) a specific surface

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of the voids, (α) greater than 500 in^2/in^3 (19.7 mm^2/mm^3), and (4) a calculated spacing factor, L, less than about 0.008 in (0.2 mm).*

The scaling observed for each of the decks is shown in Table 5 along with the estimated number of deicer applications, air void spacing factor, and indications of several construction procedures that would be expected to influence the resistance of concrete to deicers.

Also shown in Table 5 are three factors related to water-cement ratios of the concrete. The first is the water-cement ratios based upon calculations made at the time of construction. Water-cement ratios calculated from field measurements are subject to considerable variation and uncertainty. Also indicated is the use of a water-reducing admixture, which certainly would be associated with a reduced water-cement ratio. While the absorption values would reflect, among other things, the degree of consolidation, low values of absorption would also be expected to be associated with low water-cement ratios. Thus, these three factors taken together would tend to provide the best available estimate of the water-cement ratio.

The decks are listed in decreasing order of the surface scaling observed in the survey. Concrete properties that meet the ACI recommendations are designated by parentheses and generally are associated with the better levels of performance. Those which might at first glance seem to be exceptions are bridges #6 and #7, which had performed rather poorly, and bridge #8, which had performed somewhat better than might have been expected. As noted in the table, bridges #6 and #7 were the only two in the sample that had received a silicone surface treatment prior to being opened to traffic. These two bridges exhibited the greatest evidence of scaling. At the time of construction of these decks a wide variety of sealants, including silicones, were being promoted to improve the durability of concrete. Subsequent experience and studies have shown that many of these sealants are of no benefit and some, in fact, promote accelerated deterioration by maintaining a higher degree of saturation in the concrete than would be the case if the deck were left open or were treated with a "breathing" sealer such as linseed oil (Klieger and Perenchio 1963; NCHRP 1979). Similar experience has been noted with many coatings applied to masonry in buildings (Clark et als 1975). A contributing factor in the case of bridge #7 was the geometrics of the bridge, which promoted a high degree of saturation because the bridge was located at the bottom of a long vertical curve.

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* Powers (1949) originally suggested that a maximum spacing factor of 0.010 in (25mm) would provide satisfactory resistance to freezing and thawing. Subsequent work by Mielenz et al. (1958) suggested an upper limit of about 0.006 to 0.008 for extreme exposures. The value of 0.0080 was adopted by the ACI and has come to be generally used.

A further indication of the influence of the silicone treatment can be gained by comparing the performances of bridges #7 and #1 which are located on the same interstate route and contain concrete of the same materials and mixture proportions from the same ready-mixed plant. As noted in Table 5, the properties of the concrete in bridge #7 are equal to or better than those in bridge #1, which had had more applications of deicing chemicals but had performed significantly better. Bridge #1 was also better drained than bridge #7.

Bridge #8 has a considerable superelevation and has better than average drainage, which may account in part for its better than expected performance. It should also be noted that bridge #10, even with relatively light deicing, showed significant scaling, as would be expected from its properties.

Absorption will be discussed in more detail subsequently, but as previously stated is an indirect indicator of the water-cement ratio. Note that bridges #9 and #11, which were overlaid to correct deterioration of the concrete, showed very high absorption values, as did bridges #8, #10, and #17, which showed some deterioration under limited deicing.

The comparatively good performance of bridges #8 and #17 can be attributed to the limited exposure to deicing chemicals. In addition, it should be noted that the poor performance of bridge #11 probably was significantly influenced by the excessive delay in curing. On the other hand the three bridges found to have performed significantly better under heavier deicing (#1, #15, and #16) had low absorptions. While the data are limited, the two bridges with the lowest incidence of scaling both had received a linseed oil treatment. Bridge #4, which also had received a linseed oil treatment, was overlaid to correct a deficient cover over reinforcement rather than to correct deterioration of the concrete. Bridge #5 represented a specific case that will be discussed later in the report.

In summary, the scaling observed on these decks is consistent with the requirements suggested by ACI as stated earlier in the report.

Table 5

Observed Scaling, Concrete Properties, and Construction Procedures

Bridge	1977 Condition	Estimated Deicer Applications	Air Void Spacing Factor $\bar{L}$, in	Water-Cement Ratio[a]	Contained Water Reducer	Absorption of Concrete, Percent	Curing Method	Delay Between Texturing & Curing, Hours	Pro-tective Treatment
9	overlaid [b]	360	0.0100	0.47	No	6.60	WPC	0.5	None
4	overlaid	210	0.0118	0.48	No	(4.38)	WPC	0.7	Lin. Oil
11	overlaid [b]	140	(0.0062)	0.51	No	5.80	WPC	5.7	None
6	60M, 25H	468	(0.0072)	0.47	No	4.78	B&P	0.8	Silicone
7	60M, 25H	468	(.0069)	(.43)	(Yes)	(4.03)	B&P	1.2	Silicone
13	40L, 10H	650	0.0143	(.44)	(Yes)	5.61	Paper	0.8	None
10	85M, 15L	39	0.0095	0.48	No	5.40	Sand	2.5	None
5	70M, 10L	195	0.0097	0.48	No	4.56	WPC	0.5	Lin. Oil
3	40M	360	0.0099	0.46	No	5.12	WPC	0.1	None
12	30M, 30L	700	(.0064)	0.50	No	4.72	WPC	0.3	None
15	15M, 10L	660	0.0093	(.43)	(Yes)	(4.05)	WPC	0.4	None
17	10M, 25L	28	(.0075)	0.46	No	5.74	Paper	2.0	None
1	80L	702	(.0078)	(.43)	(Yes)	(4.27)	Paper	2.7	None
8	15L	168	0.0137	0.48	No	5.53	B&P	1.4	None
2	0 [c]	390	(.0048)	(.44)	No	4.78	WPC	1.7	Lin. Oil
16	0	210	0.0089	0.47	(yes)	(3.52)	WPC	0.1	Lin. Oil

Note: Curing method: WPC = white pigmented compound; B&P = burlap and polyethylene
L = light; M = medium; H = heavy (as as shown in Table 4)

a Calculated from measurements made at the time of construction
b Overlaid to correct concrete deterioration
c Condition in 1972 prior to overlaying

Penetration of Chlorides

The penetration of chlorides into concrete is a complex phenomenon. Contrary to long-held beliefs, cracks in concrete are not necessary to provide access, although cracks undoubtedly accelerate the process. Numerous studies have demonstrated that chloride will penetrate uncracked concrete, but most published studies have been unable to demonstrate clear relationships between chloride penetration and conventionally measured properties of concrete.

Ost and Monfore (1966) reported the results of laboratory tests of concrete ponded with a 2% solution of calcium chloride for 12 months. They found chloride contents by weight of cement at a depth of 1 in (25 mm) of approximately 4.0%, 7.0% and 11.0% for water-cement ratios of 0.40, 0.46, and 0.62. In currently used terms, these would translate approximately to chloride contents by weight of concrete of 9.3, 16.2 and 25.5 lb $Cl^{-}yd^{3}$ (5.5, 9.6 and 15.1 kg Cl^{-}/m^{3}) based upon the weight of concrete at these water-cement ratios. Clear and Hay (1973, 1977) reported tests of slabs exposed out of doors and ponded with 1/16 in (1.6 mm) of a 3% sodium chloride solution. The solution was renewed daily and the specimens were flushed monthly. The exposure resulted in some intermittent wetting and drying of the specimens as opposed to the continuous exposure used by Ost and Monfore. Of the numerous variables included in the study most had little, if any, effect on chloride penetration to the level of the reinforcing steel. They reported that the water-cement ratio, depth of cover, and consolidation best defined the ability of the concrete to resist chloride penetration, a finding that is consistent with the ACI recommendations stated earlier. The average chloride ion contents at a depth of 1 in (25 mm) within the concrete after 330 salt applications were 1.0, 3.6, and 5.7 lb Cl^{-}/yd^{3} (0.6, 2.1 and 3.4 kg/m^{3}) for water-cement ratios of 0.40, 0.50, and 0.60 (Clear and Hay 1973).

The average chloride ion contents at a depth of 1 in (25 mm) within the concrete after 830 cycles, given in the 1976 report by Clear and Hay, were 1.8, 11.5, 14.0 lb Cl^{-}/yd^{3} (1.1, 6.0, 8.3 kg/m^{3}) for the water-cement ratios of 0.40, 0.50, and 0.60.

The chloride contents obtained from both studies are of the same order of magnitude but for the lowest water-cement ratio, but for the higher water-cement ratios the results obtained by Clear and Hay are higher than those of Ost and Monfore. Other research has generally confirmed the higher values and has led to a general belief that "chloride penetration is inevitable." This belief has resulted in the promotion of numerous proprietary products for use as admixtures for concrete, the use of very low water-cement ratios (below 0.32), and a variety of other approaches to protecting reinforcement against corrosion.

In this light, the chloride contents obtained from this study and shown in Table 6 are of considerable interest. The chloride contents of the surfaces were variable, as would be expected, but they generally reflect the estimated deicer applications shown in Table 3. The lowest chloride contents at the surface were found in bridges #12 and #17. Bridge #17 had the lowest number of deicer applications (28), while the only exposure of the surface on bridge #12 to chlorides would be from splash and runoff from the heavily treated adjacent lane. Surface chloride ion contents greater than 0.10% by weight were found in cores from bridges #2, #3, #4, #7, #9, #10, #11, #13, and #15. With the exception of bridges #10 and #11, these values are as would be anticipated from the estimated deicer applications.

To determine the amount of intruded chlorides it is necessary to estimate the base levels of chloride in the concrete. Studies at the Research Council have indicated that some commercially available carbonate aggregates contain significant amounts of acid-leachable chlorides (Tyson 1976; Clemena and Reynolds 1980). For a specific aggregate, Tyson reported a value of 0.028% Cl^- based upon aggregate weight. Clemena and Reynolds, based upon a survey of 104 aggregate quarries, found the average acid-leachable chloride ion content for limestones and dolomites to be 0.015% while Clarke (1924) reported 0.020%. Clemena and Reynolds reported average values of 0.006% for igneous rocks and 0.004% for silicious gravels. These values agree well with values found in the bridge deck samples. Bridges #4, #5, and #16 were built with concrete containing limestone coarse aggregates from the same source. In these decks, chloride values of about 0.020% were found at a depth of 4½ in (115 mm). Bridge #10 was built with concrete containing limestone from another source which apparently contained no chlorides. Values obtained at the 4½-in (115-mm) level for concrete containing other aggregates were generally low. Thus for analysis, the chloride values obtained in this study were corrected by deducting the value measured at the 4½-in (115-mm) level, assumed to be the base value, from the value determined at the other levels.

Evaluations of the resistance of the concretes to the penetration of chlorides must be made in consideration of the high variability of the surface chlorides and also in view of the highly variable exposure of the various decks to chlorides. The use of absolute values of chlorides is misleading, since low values might indicate either good resistance to penetration or a low level of surface exposure. After several approaches were used to evaluate the data, the most appropriate seemed to be to express the chloride contents (corrected for base levels) as a percentage of the level measured at the surface. Using this approach, the distributions of chlorides in the 13 decks without overlays are illustrated in Figures 3 through 7, which present results typical of five conditions of penetration. The results for all samples were given in Table 6.

Figure 3 illustrates the case of comparatively rapid entry observed on bridge #13. This behavior was also observed in varying degrees on bridges #3, #7, #8, and #10. This type of behavior would be expected on the assumption that "chloride penetration is inevitable" and applies to almost half of the 17 decks. The results shown in Figure 4 for bridge #5 indicate an extremely variable performance for the two samples from the same deck. Sample 1 showed comparatively good resistance to penetration and sample 2 showed a very rapid ingress of chloride. No other deck showed performance of this type, and the causes are discussed later in the report.

Figure 4 illustrates very good resistance to chloride penetration exhibited by bridge #15, which is the performance desired. Similar behavior was evidenced in bridges #1 and #6. The results shown in Figure 6 for bridge #16 illustrate the same good resistance to chloride penetration as that illustrated in Figure 5 below the 3/4-in (10-mm) level, even though the Cl^- contents are quite high, albeit variable, above that level. The difference between the chloride contents of the two samples undoubtedly results from the fact that sample 2 was located 6 ft (1.3 m) from the curb in a lane built to accommodate future widening of the approach pavement and receives little traffic during snow but permits the accumulation of snow containing chloride with intermittent melting and freezing, whereas sample 1, 17 ft (5.2 m) from the curb was located approximately at the center of the traffic lane. The total width of this bridge was 54 ft (16.5 mm). The distribution of chlorides shown in Figure 7 for bridge #17 is as would be expected in the absence of significant salt applications, as was also true for bridge #12.

Table 6

Chloride Content and Absorption of Cores Removed in 1977

	Chloride Content, % Cl^- by Weight of Concrete				Absorption, % by Weight of Concrete		
Core	Surface	3/4 in. (19mm)	1 3/4 in. (44mm)	4½ in. (115mm)	2-4 in. (50-100mm)	¼-½ in. (6-13mm)	Upper-Lower
1-1	0.077	0.006	0.005	0.004	4.42	4.44	+0.02
1-2	0.089	0.017	0.007	0.004	4.43	5.26	+0.83
2-1	0.140	0.113	0.077	0.018	4.65	5.38	+0.73
2-2	0.067	0.047	0.023	0.015	4.50	4.65	+0.15
3-1	0.158	0.118	0.040	0.004	5.36	5.10	-0.26
3-2	0.263	0.187	0.118	0.005	4.05	5.71	+1.66
4-1	0.113	0.087	0.041	0.020	4.75	3.56	-1.18
4-2	0.167	0.130	0.105	0.021	5.03	5.26	+0.23
5-1	0.081	0.039	0.021	0.025	4.68	4.66	-0.02
5-2	0.212	0.189	0.155	0.032	4.76	5.05	+0.29
6-1	0.081	0.032	0.007	0.003	4.51	4.58	+0.07
6-2	0.059	0.018	0.003	0.003	5.16	4.85	-0.31
7-1	0.114	0.057	0.017	0.004	5.05	5.15	+0.10
7-2	0.196	0.078	0.031	0.004	4.94	5.12	+0.18
8-1	0.091	0.050	0.035	0.006	5.22	5.02	-0.20
8-2	0.054	0.050	0.029	0.006	5.61	5.11	-0.50
9-1	0.062	0.125	no sample	no sample	5.81	5.46	-0.35
9-2	0.143	0.191	0.93	0.006	6.84	6.69	-0.15
10-1	0.091	0.092	0.047	0.007	5.96	4.84	-1.12
10-2	0.128	0.057	0.033	0.011	6.10	4.68	-1.42
11-1	0.119	0.060	0.043	0.006	5.01	6.19	+1.18
11-2	0.111	0.143	0.70	0.013	6.37	5.64	-0.73
12-1	0.011	0.006	0.003	0.004	4.79	5.05	+0.26
12-2	0.019	0.014	0.003	0.003	4.01	4.60	-0.50
13-1	0.190	0.131	0.062	0.004	6.10	5.75	-0.35
13-2	0.138	0.088	0.025	0.004	4.71	6.03	+2.28
14-1	overlaid prior to traffic (See Figure 33)						
14-2							
15-1	0.103	0.007	0.005	0.003	4.49	broke	---
15-2	0.130	0.012	0.004	0.004	3.43	broke	---
16-1	0.073	0.032	0.027	0.028	3.72	2.19	-1.53
16-2	0.236	0.108	0.020	0.027	3.88	4.50	+0.62
17-1	0.12	0.007	0.005	0.005	6.33	6.12	-0.21
17-2	0.013	0.007	0.004	0.005	4.97	5.53	+0.56

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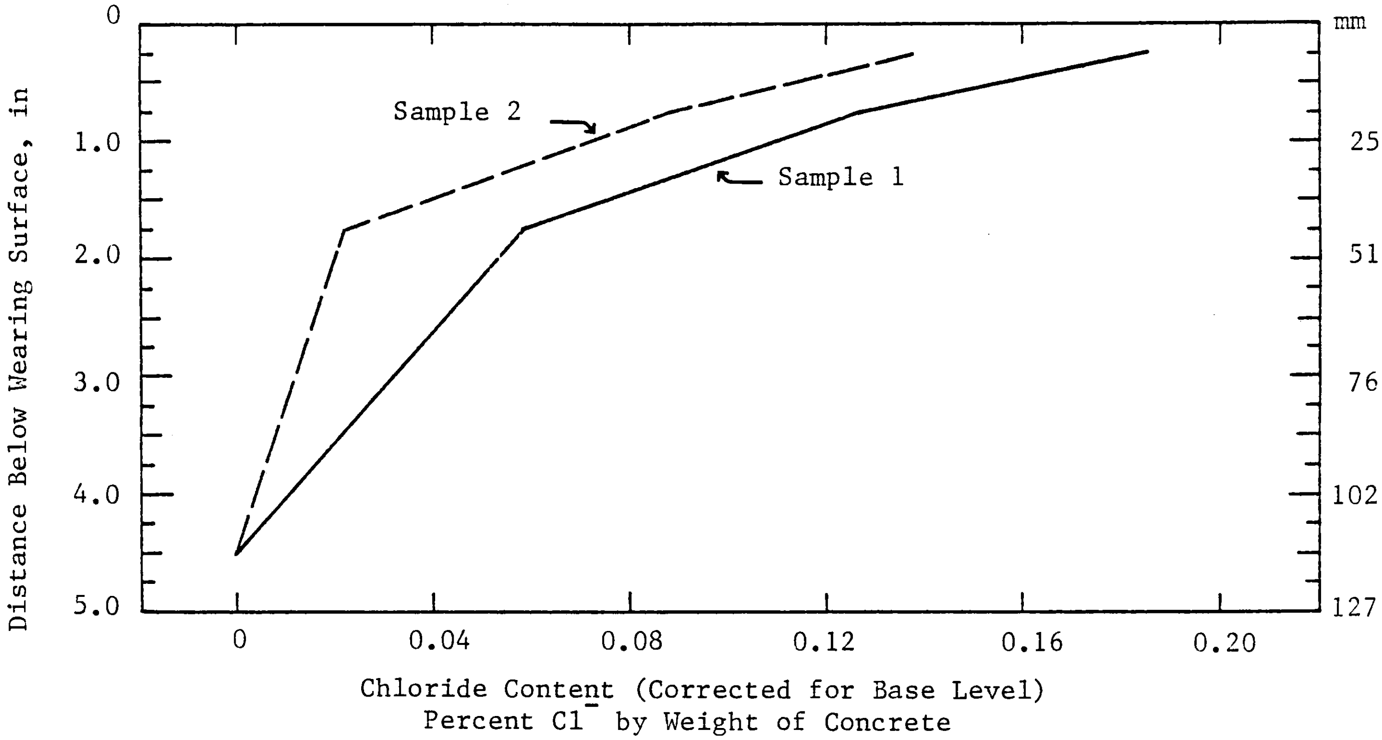


Figure 3. Chloride distribution in bridge #13. Relationship typical of bridges #3, #7, #8, #10 and #13.

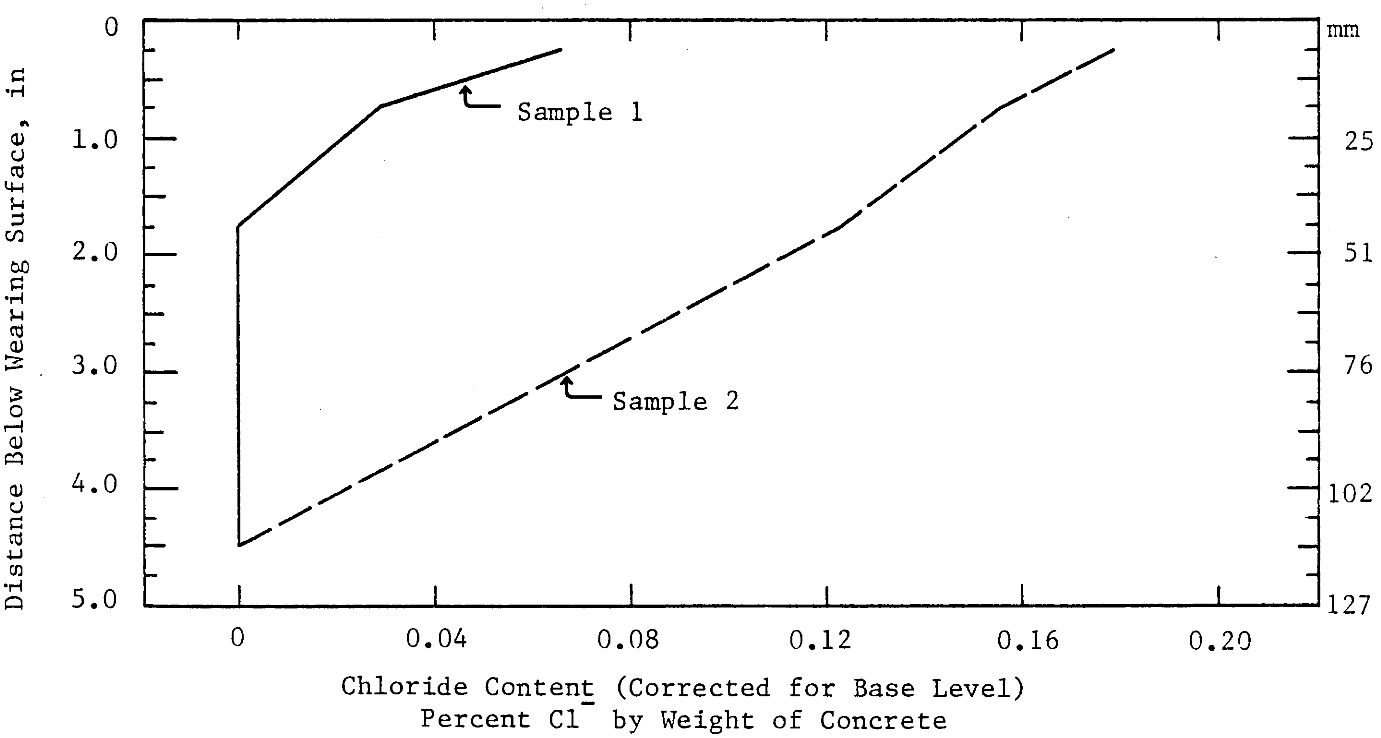


Figure 4. Chloride distribution in bridge #5 samples.

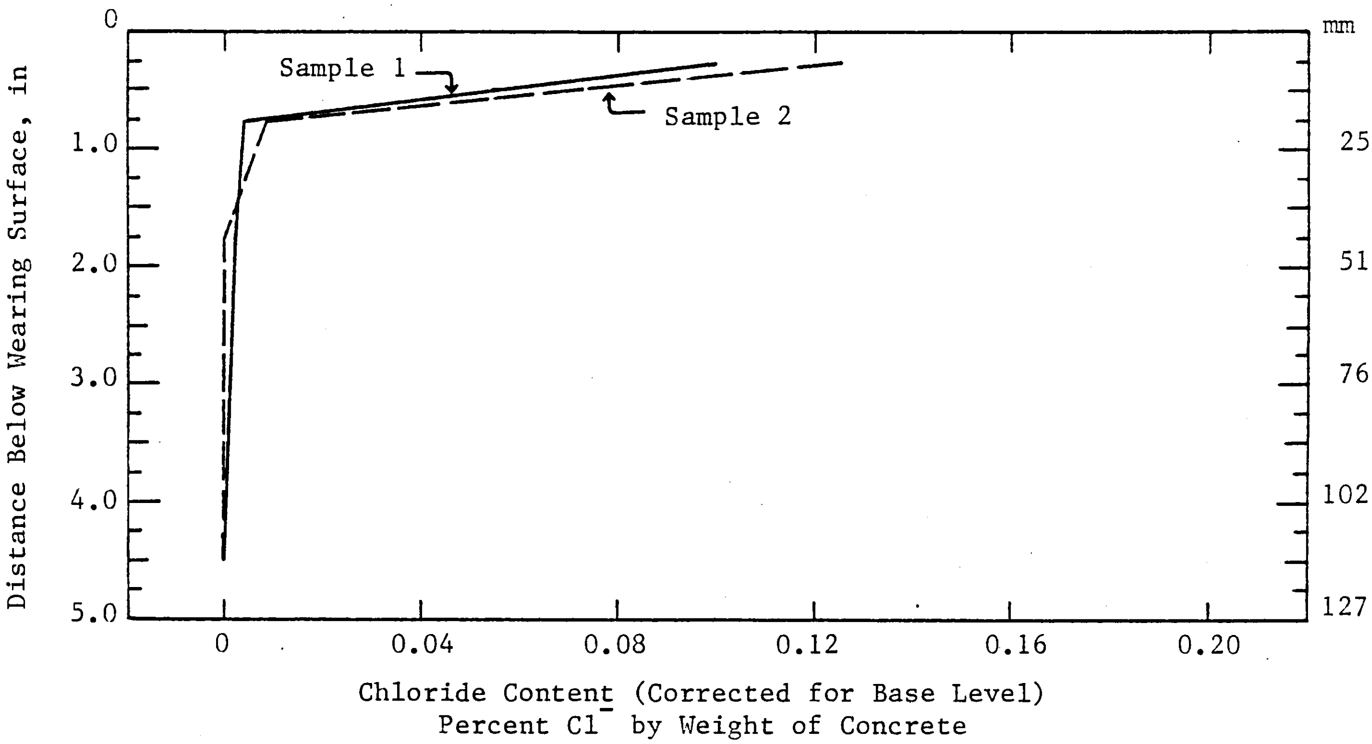


Figure 5. Chloride distribution in bridge #15. Relationship typical of bridges #1, #6 and #15.

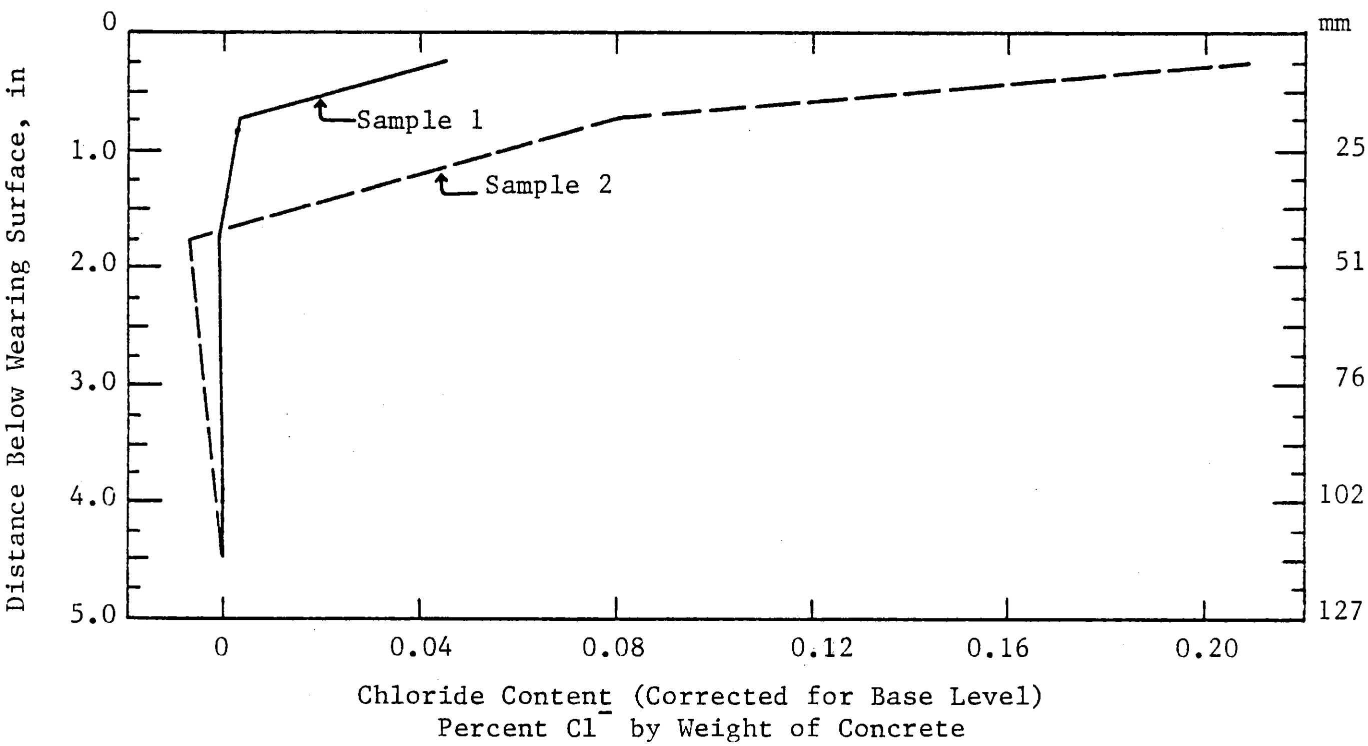


Figure 6. Chloride distribution in bridge #16.

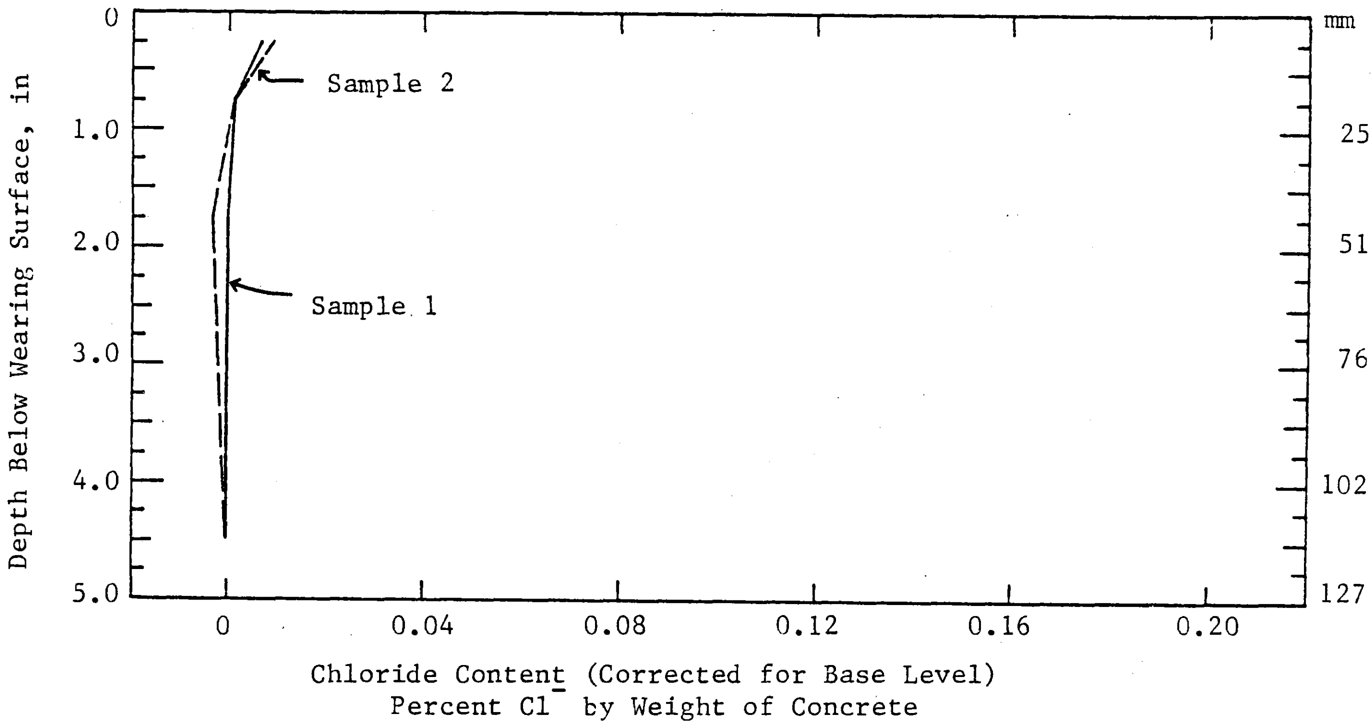


Figure 7. Chloride distribution for bridge #17 (typical for bridges #12 and #17).

In addition to being influenced by the properties of the concrete, the penetration of chloride is a function of the environment of the samples. This is illustrated in Figure 8, which presents the chloride content at the 3/4-in. (10-mm) level as a function of distance from the wheel guard on the right side (travel lane) of the seven uncovered spans that carry traffic in one direction. A line connects the two samples from the same span. There is a general reduction in chloride levels with increasing distance from the curb line. This would be expected on the basis of drainage toward the curbline. At first glance, this relationship may not be clear since there are two cases (bridges #7 and #16) where the chloride content appears to increase with distance from the wheel guard, contrary to what might be expected due to draining if not on a superelevated deck. On closer examination these two apparent anomalies actually strengthen the relationship since in the case of bridge #7 the traffic volume and the relative narrowness of this interstate structure result in a very large volume of traffic in what would normally be the passing lane, with the result that it is similar to a bridge carrying two-way traffic. If the distance to the nearest curb is used, as shown by the dotted line in the figure, the expected relationship is observed. As noted earlier, bridge #16 contains a lane, or shoulder, S, for future widening, and this lane nearest the curb does not carry any traffic and has very little surface chloride. The apparent anomalies, then, are related to the location of the vehicles as indicated in the T (traffic) or P (passing lanes), or on the S (shoulder), as well as the distance from the curb nearest to the traffic. It is interesting also to note that for the samples from bridge #6, the only condition where both samples were in the passing lane, the chlorides for both samples were comparatively low. The very great difference between the two cores from bridge #5 is related to the properties of the concretes, as will be discussed later. The data confirm that chloride concentrations are related to the drainage characteristics of the deck.

Because of the differences observed in the ratios of penetration of chlorides among the decks as illustrated in Figures 3 through 7, it is of value to seek relationships among the reductions in chloride contents with increasing depth from the surface and the properties of the concrete as reflected by characteristics of the concrete and observations made during construction. The reduction of chloride penetration and several concrete parameters are shown in Tables 7 and 8 for depths of 3/4 in (19 mm) and 1 3/4 in (44 mm), respectively. The concrete properties meeting ACI recommendations are indicated by parentheses.

Figure 9 shows the relationship between the reduction of chloride at the 1 3/4-in (44-mm) level as compared with the surface and the total water absorption of the cores after 528 hours of immersion and oven drying. In Figure 9 the ratio of the chloride content at 1 3/4 in (44 mm) to that at the surface is plotted for each core as a function of the absorption determined on the portion between 2 and 4 in (50-100 mm). Except for cores 3-2, 5-2, 10-2, and possibly 6-2, a surprisingly good relationship exists, which shows that for absorption values of 4.5% and below, the intrusion of chlorides to a depth of 1 3/4 in (44 mm) has been significantly retarded or eliminated, whereas above this absorption value the intrusion increases with absorption values above 4.5%.

Absorptions were also determined on the portion of the core between ¼ - ½ in (6-12 mm), designated as segment 2 in Figure 1. These results are included in Table 7 for the 10 bridges that remained uncovered at the time of the 1977 survey, with the exception of bridges #12 and #17, which had received minimal direct deicing as indicated in Figure 7.

Although these portions were very small, the data suggest that the anomalous results are related to variations of absorption vertically within the core. In Figure 9, solid circles designate samples for which the absorption of upper portions of the core (above the chloride sample) were higher than that of the portion below the location of the chloride sample. The differences in absorption results are also indicated in

Figure 9. Data points indicated by triangles represent the reverse condition. The effect would be to shift the circles to the right and triangles to the left, thus strengthening the relationship.

From the data shown in Figure 9, it can be concluded that chloride intrusion is denied at a depth of 1 3/4 in (44 mm) if the absorption of the concrete is below about 4.5%.

The relative amounts of chloride at a depth of 3/4 in (19 mm) are plotted in the same manner in Figure 10. The results are also shown in Table 8. There is more variability, as would be expected because of the surface variability, but the trends are the same as for the deeper level. These data indicate that intrusion of chlorides is effectively denied at a depth of 3/4 in (19 mm) when the absorption of the concrete is below about 3.5% to 4.0%. Similar results have been reported recently by Leslie and Chamberlin (1980), but the concretes that they studied had absorption values in a much smaller range than those in this study.

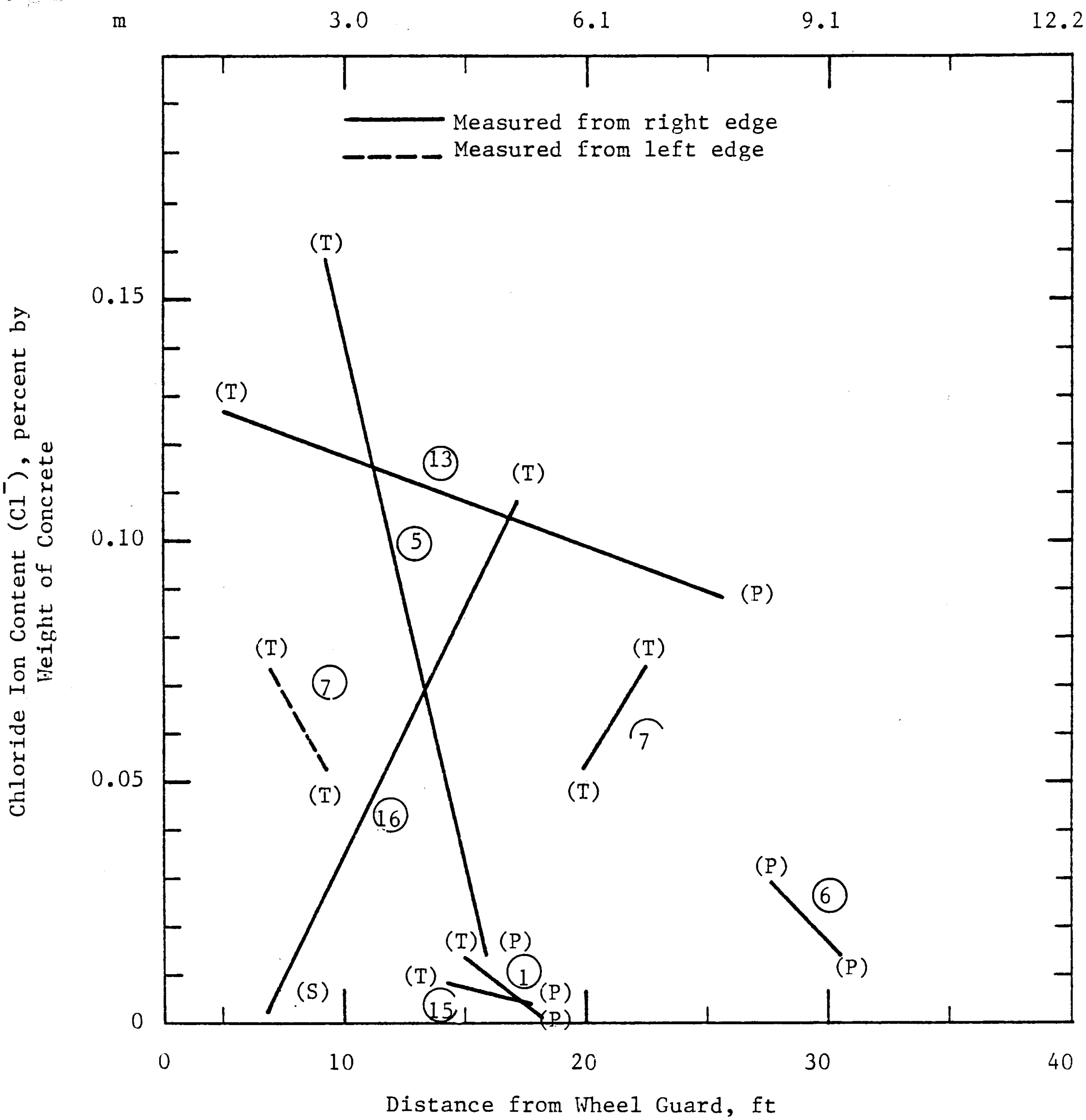


Figure 8. Chloride contents are related to location on the deck and traffic location.

Table 7

Relationships Between Chloride Reduction in Top 1 3/4 in (44 mm) and Concrete Parameters

Sample	Ratio of Cl^- 1 3/4"in (44mm) Surface)	Water-Cement Ratio	Water Reduction	Spacing* Factor, $\overline{L}$, in Total Upper 1½"(38mm)	Absorption Percent 2-4 in (50-100mm)	Spacing* Factor, $\overline{L}$, in Surface	Absorption Percent, ¼-½ in. (6-13mm)
16-2	0	.47	(Yes)	.0102	(3.88)	.0096	(4.5)
16-1	0	.47	(Yes)	.0084	(3.72)	.0081	(2.19)
15-2	0	(.40)	(Yes)	(.0066)	(3.43)	(.0052)	Broke
6-2	0	.47	No	(.0077)	5.16	(.0080)	4.85
5-1	0	.48	No	(.0102)	4.68	(.0122)	4.66
1-1	.01	(.42)	(Yes)	(.0078)	(4.42)	No Sample	(4.44)
1-2	.01	(.45)	(Yes)	(.0072)	(4.43)	(.0080)	5.26
15-1	.02	.46	(Yes)	(.0102)	(4.49)	.0084	Broke
6-1	.05	.47	No	(.0071)	4.51	(.0077)	4.58
7-1	.12	(.43)	(Yes)	(.0056)	5.05	(.0052)	5.15
7-2	.14	(.42)	(Yes)	(.0079)	4.94	(.0080)	5.12
13-2	.16	(.44)	(Yes)	.0160	4.71	.0144	6.03
10-2	.19	.47	No	.0094	6.10	.0083	4.68
3-1	.26	.46	No	.0106	5.36	(.0107)	5.10
13-1	.31	(.44)	(Yes)	.0096	6.10	.0096	5.75
8-1	.34	.48	No	.0145	5.22	.0156	5.02
10.1	.48	.49	No	.0097	5.96	.0089	4.84
8.2	.48	.48	No	.0130	5.61	.0116	5.1
3-2	.50	.46	No	.0099	(4.05)	.0096	5.71
5-2	.68	.48	No	.0099	4.76	.0104	5.05

* to convert $\overline{L}$ in inches to mm multiply by 25.4
( ) values meet ACI Guidelines

Table 8

Relationships Between Chloride Reduction at Depth of 3/4 in (19 mm) and Concrete Parameters

Sample	Ratio of Cl^- 3/4 in (19mm) Surface	Water-Cement Ratio	Water Reduction	Spacing* Factor, $\bar{L}$, in, Overall	Absorption Percent 2-4 in (50-100mm)	Spacing* Factor, $\bar{L}$, in. Surface	Absorption Percent ¼-½ in (6-13mm)
1-2	.02	(.45)	(Yes)	(.0072)	(4.43)	(.0080)	5.26
1-1	.03	(.42)	(Yes)	(.0078)	(4.42)	No Sample	(4.44)
15-1	.04	.46	(Yes)	(.0102)	(4.49)	.0084	Broke
15-2	.06	(.40)	(Yes)	(.0066)	(3.43)	(.0052)	Broke
16-1	.09	.47	(Yes)	(.0084)	(3.72)	(.0081)	(2.19)
5-1	.25	.48	No	(.0102)	4.68	(.0122)	4.66
6-2	.26	.47	No	(.0077)	5.16	(.0080)	4.85
6-1	.37	.47	No	(.0071)	4.51	(.0077)	4.58
7-2	.38	(.42)	(Yes)	(.0079)	4.94	(.0080)	5.12
16-2	.38	.47	Yes	.0102	(3.88)	(.0096)	(4.50)
10-2	.39	.47	No	.0094	6.10	.0083	4.68
7-1	.48	(.43)	(Yes)	(.0056)	5.05	(.0052)	5.15
8-1	.52	.48	No	.0145	5.22	(.0156)	5.02
13-2	.63	(.44)	(Yes)	.0160	4.71	.0144	6.03
13-1	.68	(.44)	(Yes)	.0096	6.10	.0096	5.75
3-2	.71	.46	No	.0099	(4.05)	.0096	5.71
3-1	.74	.46	No	.0106	5.36	.0107	5.10
5-2	.87	.48	No	.0099	4.76	.0104	5.05
8-2	.91	.48	No	.0130	5.61	.0116	5.11
10-1	1.01	.49	No	.0097	5.96	.0089	4.84

* to convert $\bar{L}$ in inches to mm multiply by 25.4

( ) values meet ACI Guidelines

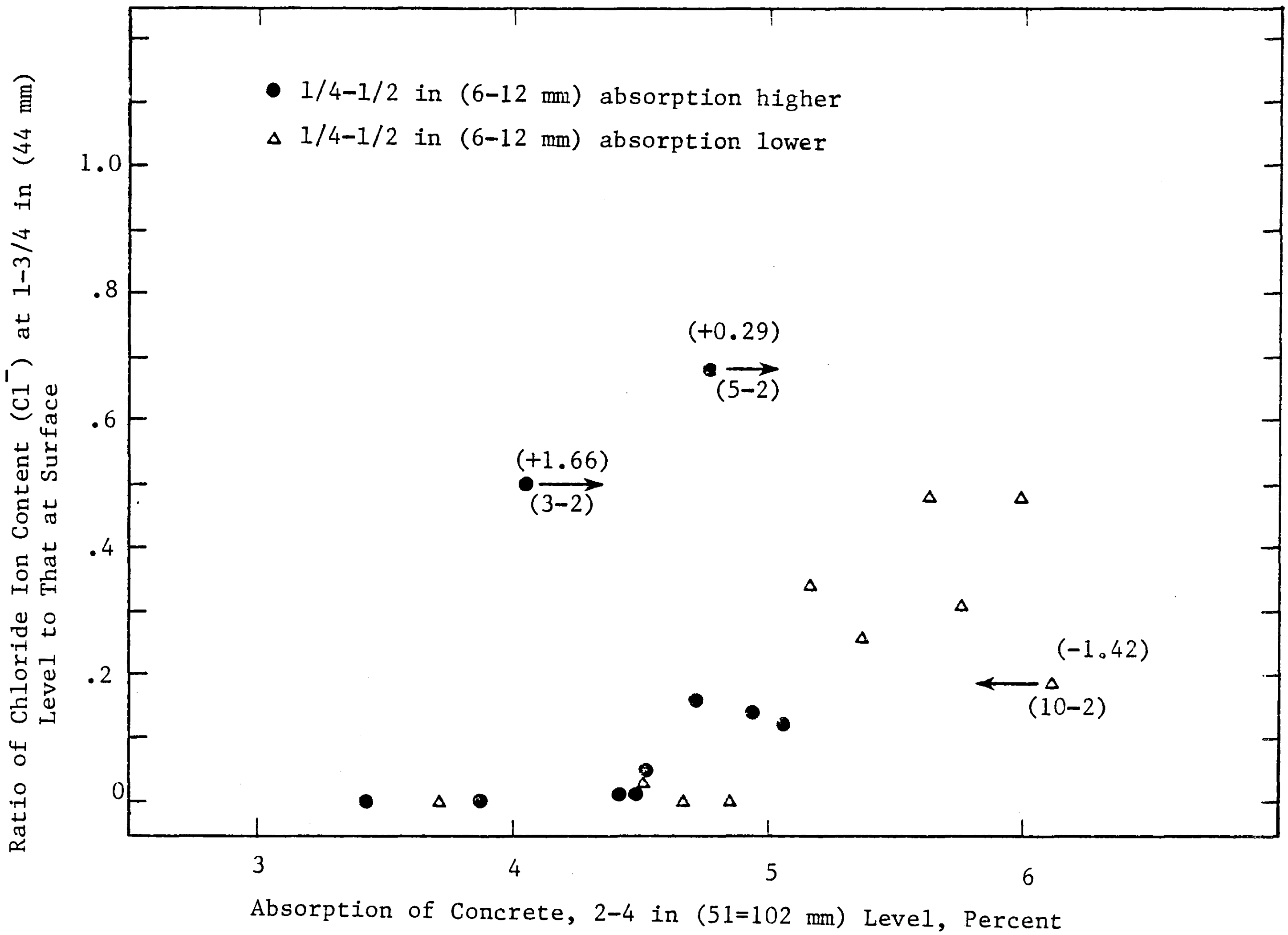


Figure 9. Relation of chloride intrusion to a depth of 1 3/4 in (44 mm) and absorption of the concrete.

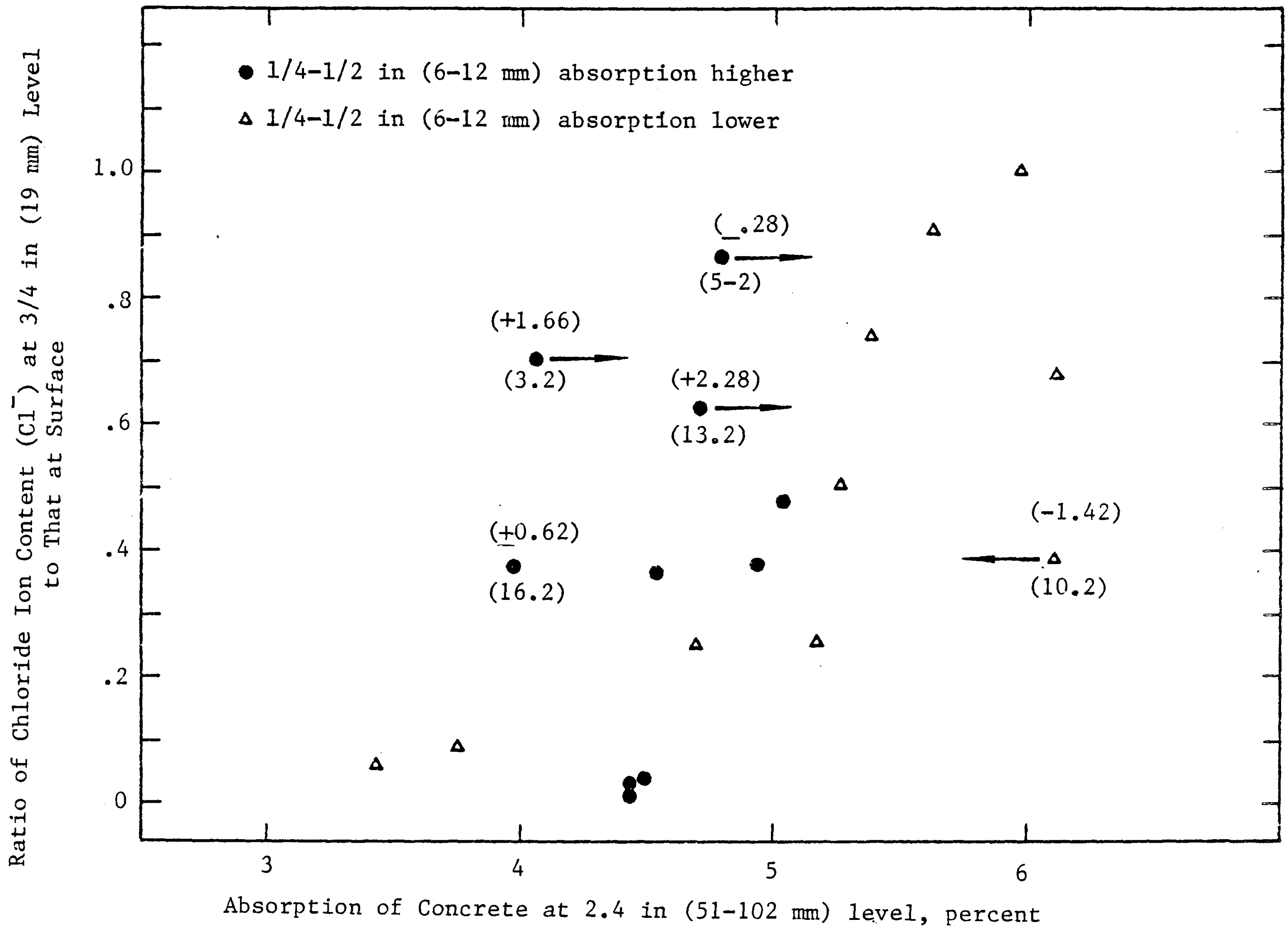


Figure 10. Relationship of chloride intrusion ¼ - ½ in (6-13mm) absorption higher to a depth of 3/4 in (19mm) and ¼ - ½ in (6-13mm) absorption lower.

Also included in Tables 7 and 8 are the water-cement ratios calculated from the 1963 field data and an indication as to whether or not the concrete mixture contained a water reducing admixture. As indicated in the original report (Newlon 1971) and discussed earlier, the calculated water-cement ratios are not completely reliable, but it would be expected that concretes containing the admixtures would have lower water contents than those without. In addition to absorption values, the spacing factors measured and calculated in accordance with ASTM Recommended Practice Method C-457 "Microscopical Determination of Air-void Content and Parameters of the Air-void System in Hardened Concrete" are shown. Two spacing factors are shown: one for the upper surface of the companion core and one based upon all of the surfaces observed.

In Tables 7 and 8 the samples are listed in order of increasing chloride ratios; i.e., by increasing chloride penetration for depths of 3/4 in (19 mm) and 1 3/4 in (44 mm), respectively. The lowest ratio in Table 8 (0.02) means that 98% of the surface chloride did not reach the indicated depth.

In addition to the relationship shown in Figures 9 and 10, comparisons of the data in Tables 7 and 8 indicate that a reduction of chloride intrusion tends to be associated with the following characteristics: (1) absorption less than 4.5%, (2) spacing factors less then 0.008 in (0.20 mm), and (3) low water-cement ratios (as indicated by reported water-cement ratios, absorption values, or presence of a water reducing admixture).

Absorption, porosity, and permeability are interrelated but not always directly. For example, materials may have a high porosity but low permeability. Absorption is more directly related to permeability than to porosity, since absorption and permeability depend not only on the volume of voids but also on the interconnection of these voids. Primary factors influencing absorption, permeability, or both are the water-cement ratio, cement-aggregate ratio, aggregate gradings, air entrainment, consistency, degree of consolidation, and adequacy of curing. Excessive water-cement ratios are particularly important because of the increase in bleeding channels, etc., that promote the ingress of waterborne chlorides. In view of the many interacting factors, the good relationships shown in Figures 9 and 10 are in some respects surprising.

The penetration of chlorides into concrete is complex and occurs by several processes. Chlorides can enter the concrete dissolved in bulk water. This mode would be expected to be related to absorption. Chloride ions can also migrate in continuously saturated concrete by complex mechanisms. In recent years research has concentrated on this latter mechanism, which has been shown to be a function of not only permeability and the capacity of the concrete for chloride binding but also of the ion exchange capacity of the system. Certain cements have been shown to react chemically with entering chlorides to a greater extent than others. Since the cements used in the decks studied in this project were all type II with rather similar compositions, cement composition should not have a significant influence on the results.

As discussed earlier, the deicing chemicals were mixtures of various chloride-bearing components. It has also been shown that the chloride penetration is substantially higher from a solution of calcium chloride than from a solution of sodium chloride of the same concentration (Theissing et al. 1975, cited by Gjorv and Vennesland 1979). It should be noted that this finding is incompatible with the results obtained in the separate studies by Ost and Monfore (1966) and Clear and Hay (1973) cited earlier.

It is believed that the major causes of the relationships shown in Figures 9 and 10 are the water-cement ratio and, to a lesser extent, the degree of compaction of the concretes. The water-cement ratio is one of the most important properties of concrete, but unfortunately one of the most difficult to measure accurately in the field because of inaccuracies in determinations of aggregate moisture contents, unacknowledged general or localized retempering, etc. As noted in the original report (Newlon 1971) based upon the data available, the water-cement ratios for the concretes intended to be the same were indicated to vary by as much as 0.40 - 0.51. It is probable that the variation was greater in localized areas. The relationship between the water-cement ratio and 24-hour absorption determined in 1964 is emphasized in Figure 11, which is repeated from the original report. Those concretes containing a water reducing admixture showed a significantly lower absorption than those that did not, despite the similarity of the calculated water-cement ratios. Thus, it appears that for the conditions to which these decks have been exposed, the water-cement ratio and other factors that contribute to an absorption of about 4.5% or below have contributed to significantly reduced rates of chloride intrusion. Other factors will be discussed later in the section on petrographic studies.

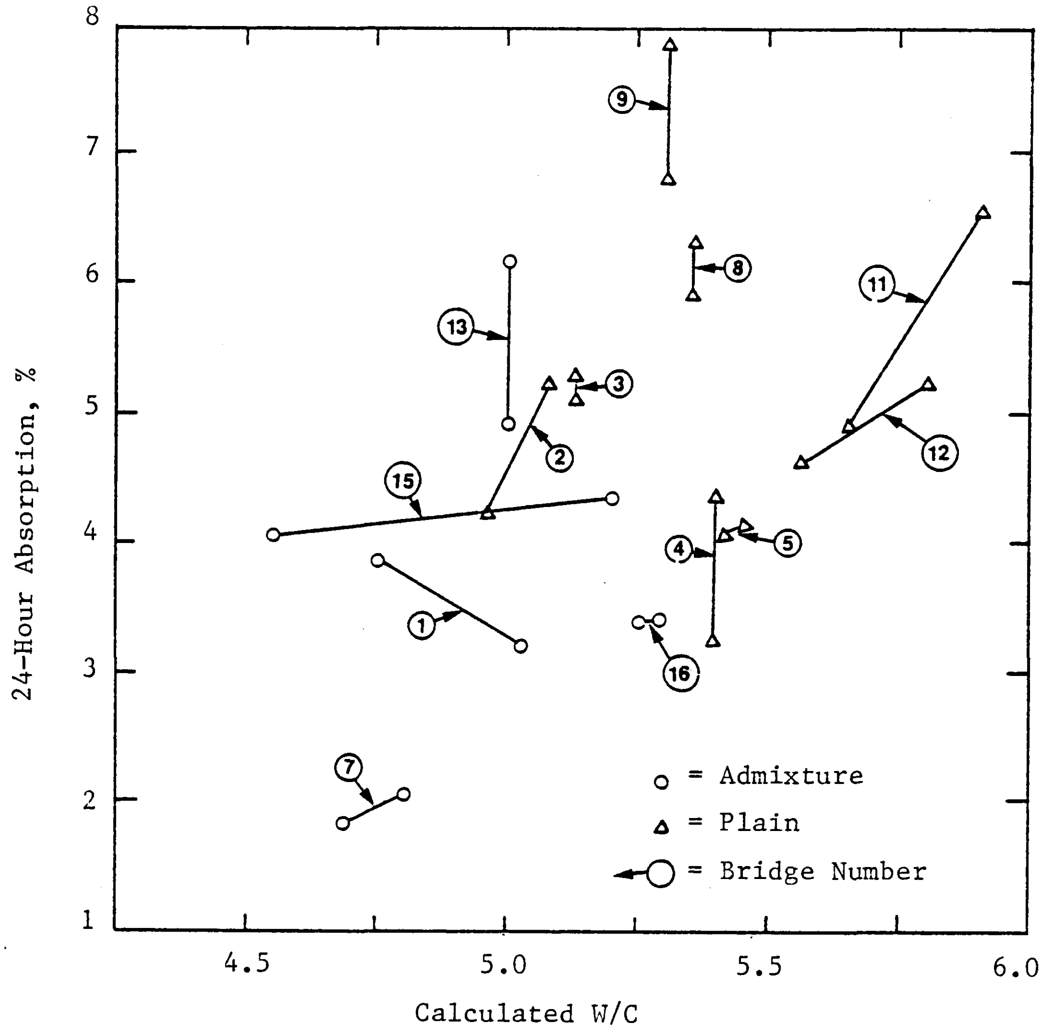


Figure 11. Relationship between measured absorption and calculated water-cement ratio. Data not available for bridges #6, #10, #14, and #17. (Reproduced from Newlon 1971.)

Corrosion Potential

Half cell potential measurements indicate the potential for corrosion activity in a bridge deck. The development of the method and the significance of the results have been extensively studied and summarized (NCHRP 1979). The numerical value of the potential increases with an increase in the amount of corrosion, but the potential is not a measure of the rate of corrosion. The greater the area of active potentials, however, the greater will be the probable amount of corrosion.

Corrosion potentials were determined in accordance with ASTM C-876 "Half Cell Potentials of Reinforcing Steel in Concrete" using a copper/copper sulfate half cell. In all cases, one lead of the voltmeter was connected directly to reinforcing steel and a 5-ft (1.5-m) grid was used.

Convention dictates that the potential be reported as negative.

The corrosion potential results are shown in Figures 12-28. In each figure the data are summarized in two ways: (1) a cumulative frequency diagram based upon individual measurements, and (2) areas of corrosion potential representing the three ranges conventionally defined in interpreting results. These three areas have been defined by Clear and Hay (1973), as follows:

a. Values less negative than -0.20V CSE indicate that there is a 90% probability that the reinforcing steel is not corroding.

b. Values in the range -0.20 to -0.35 V CSE indicate uncertain corrosion activity.

c. Values more negative than -0.35 V CSE indicate that there is a 90% probability that corrosion of the reinforcing steel is occurring at the time of measurement.

The measurements were made beginning at one wheel guard and progressing across the bridge as far as traffic conditions would permit under the traffic control used. Thus, the entire width of deck surface was not covered in all cases; however, the entire length was. The test areas covered, along with the core sample locations, are indicated in the figures. Comparison of these figures with those in Appendix A provides the overall context for the locations. The corrosion potential values shown on the grid represent the values taken at the corner of the area from which the measurements started. For example, in Figure 12 (bridge #1) the values represent the point at the upper right of the block, while in Figure 13 (bridge 2), the values represent the point at the lower left of the block, etc. Unshaded areas occur when the entire width of the deck was not tested.

On 5 of the 17 decks (bridges #1, #8, #15, #16, and #17) 90% of the readings were below -0.20 V CSE, which indicates no active corrosion. On 3 of these decks (#8, #15, and #16), there were small areas with readings between -0.20 and -0.35 V CSE. On decks #2, #12, and #14, two-thirds of the readings were more negative than -0.35 V CSE, which indicates active corrosion. On the remaining 9 decks, 90% of the readings were below a value in the range between -0.20 and -0.35 V CSE, which indicates an uncertain condition. Of the 3 decks indicating active corrosion, 2 had been overlaid; #14 for its entire service life and #2 for 5 years.

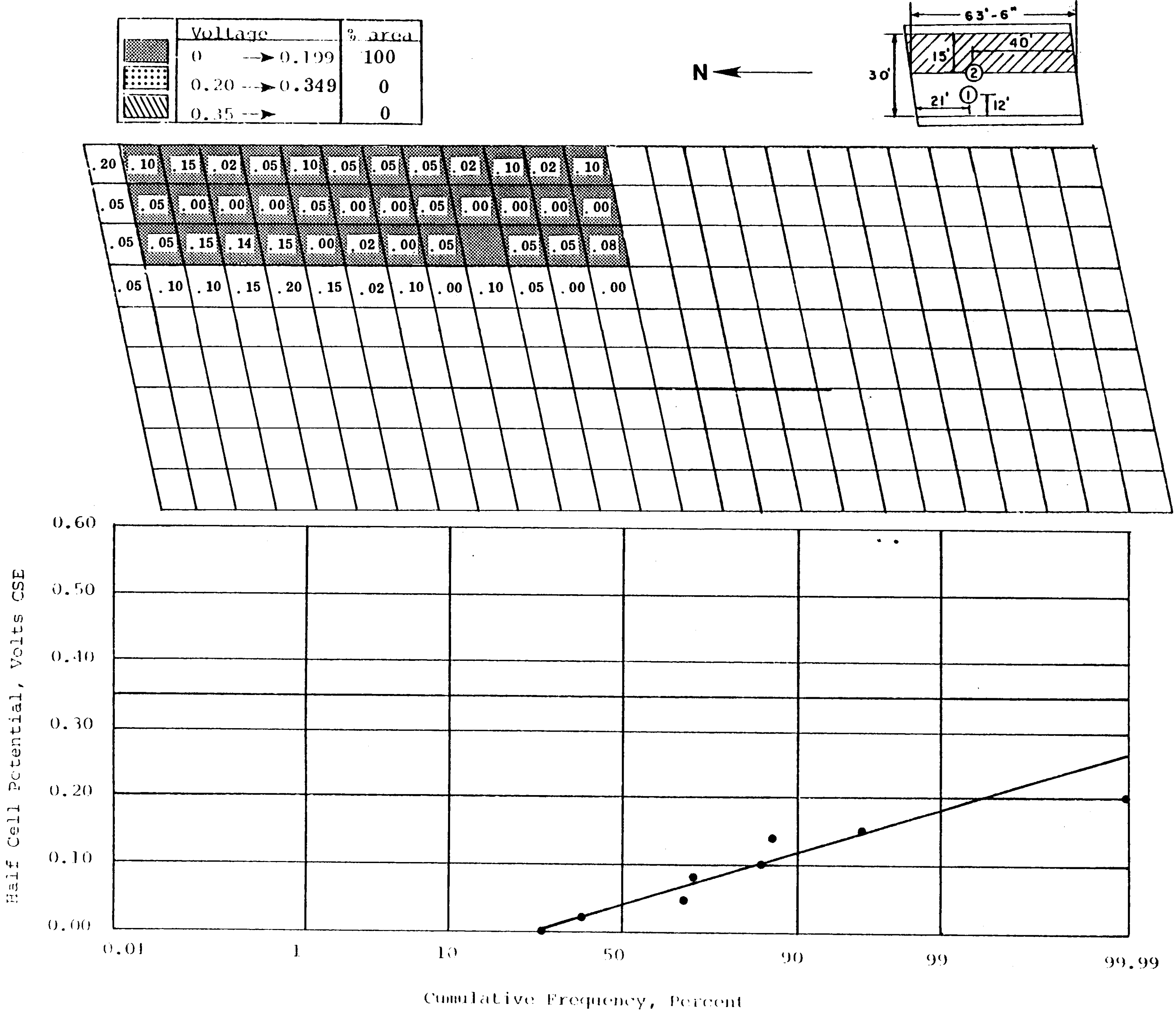


Figure 12. Corrosion potentials for bridge #1. All values in V-CSE.

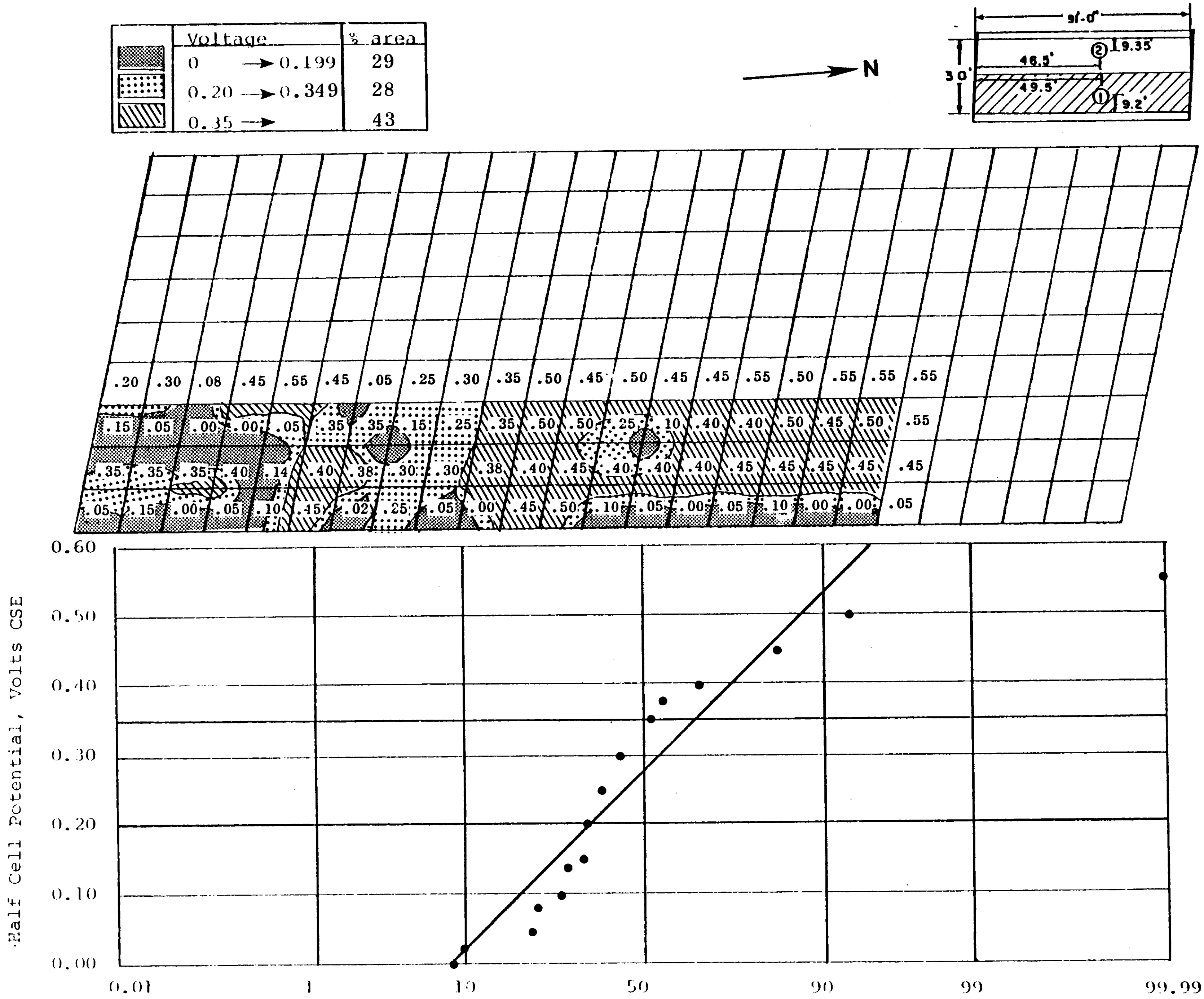


Figure 13. Corrosion potentials for bridge #2. All values in V-CSE.

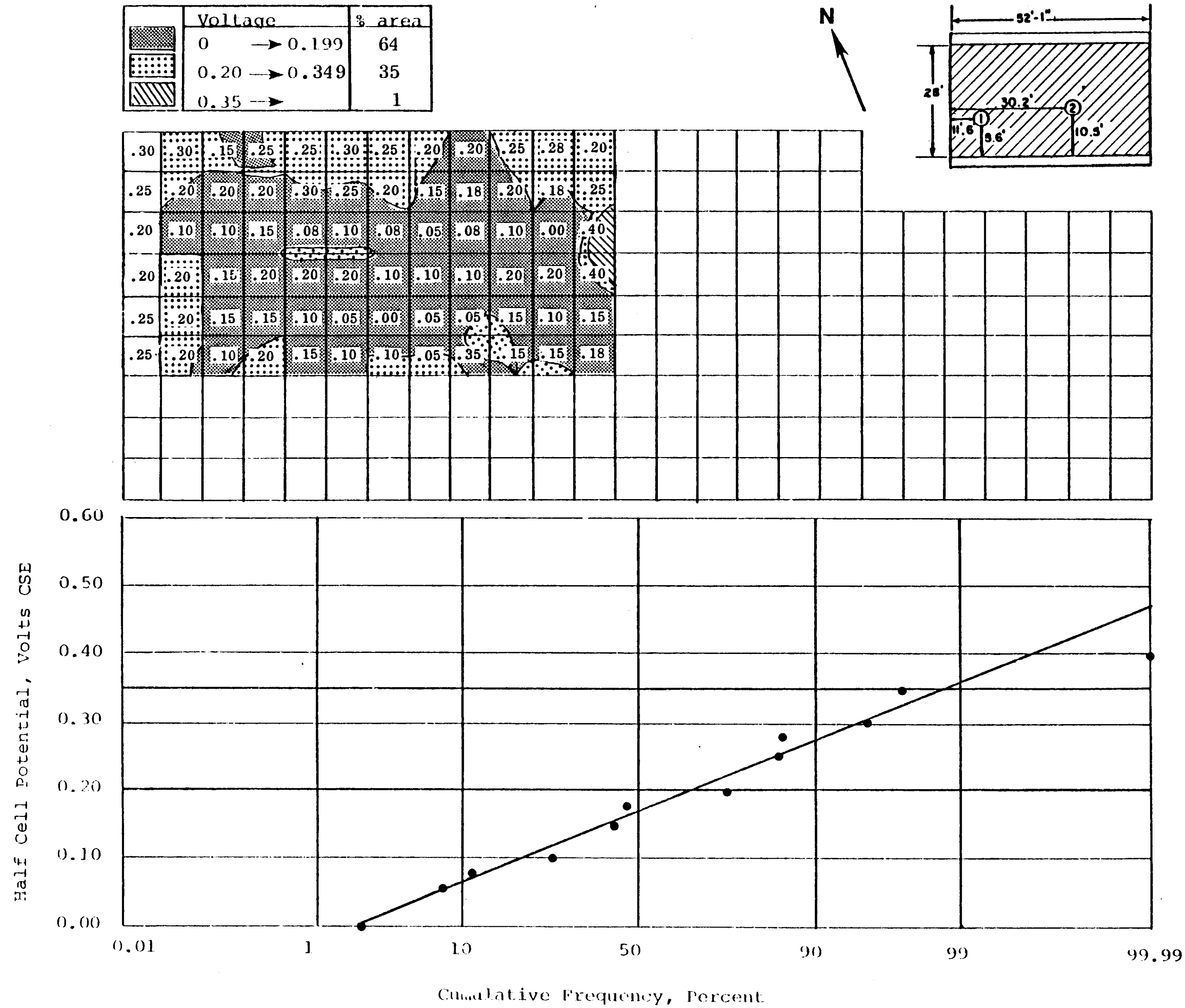


Figure 14. Corrosion potentials for bridge #3. All values in V-CSE.

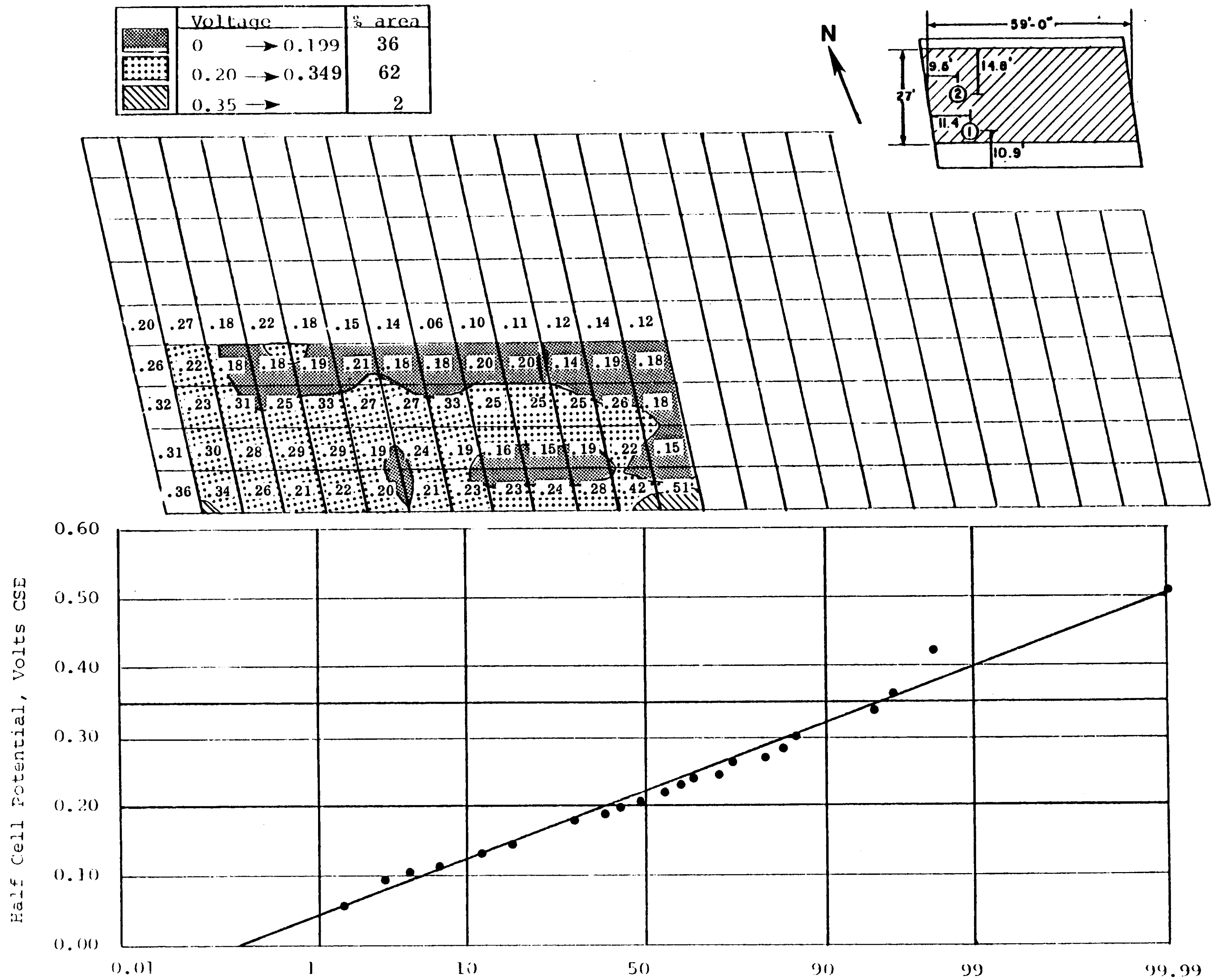


Figure 15. Corrosion potentials for bridge #4. All values in V-CSE.

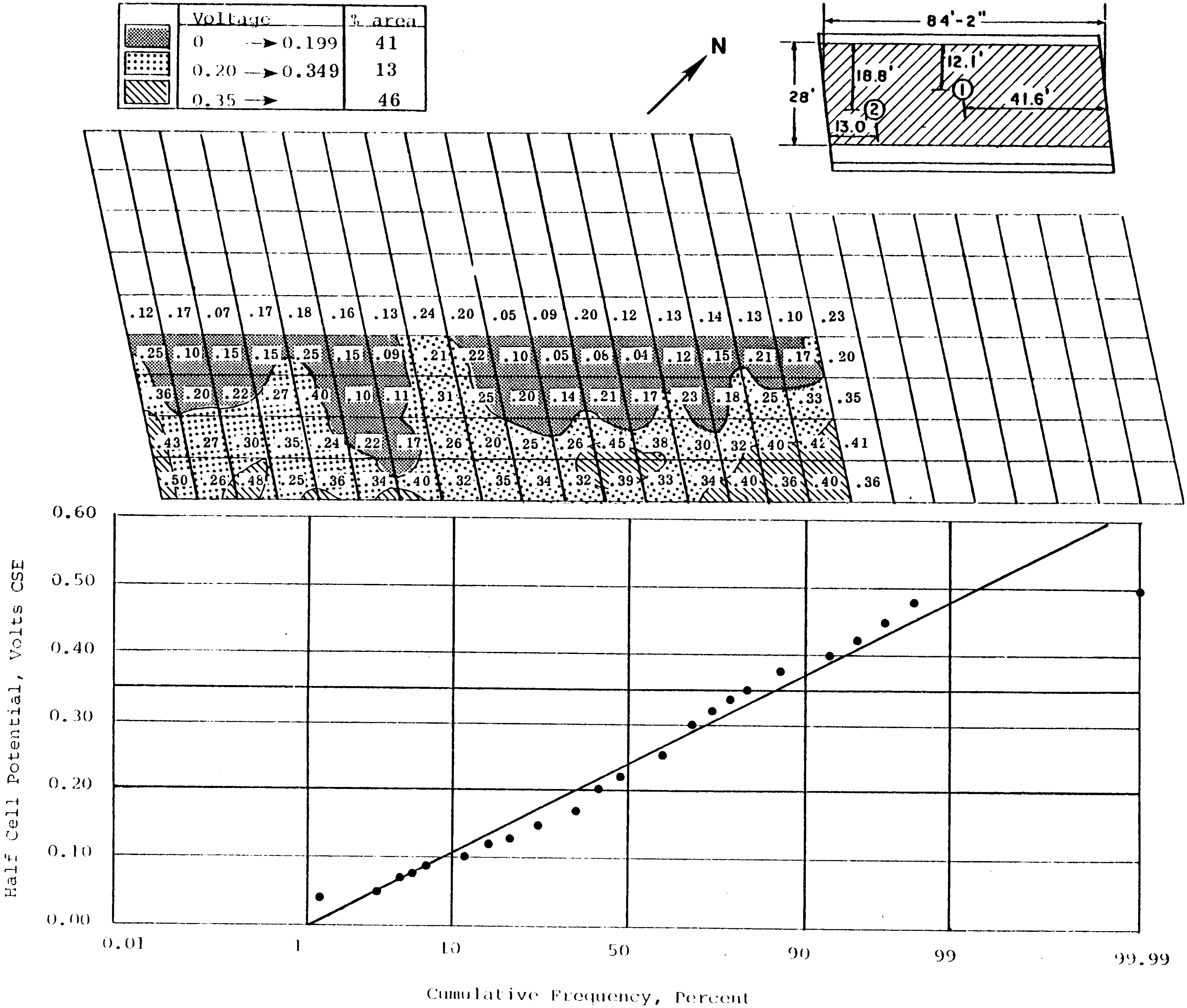


Figure 16. Corrosion potentials for bridge #5. All values in V-CSE.

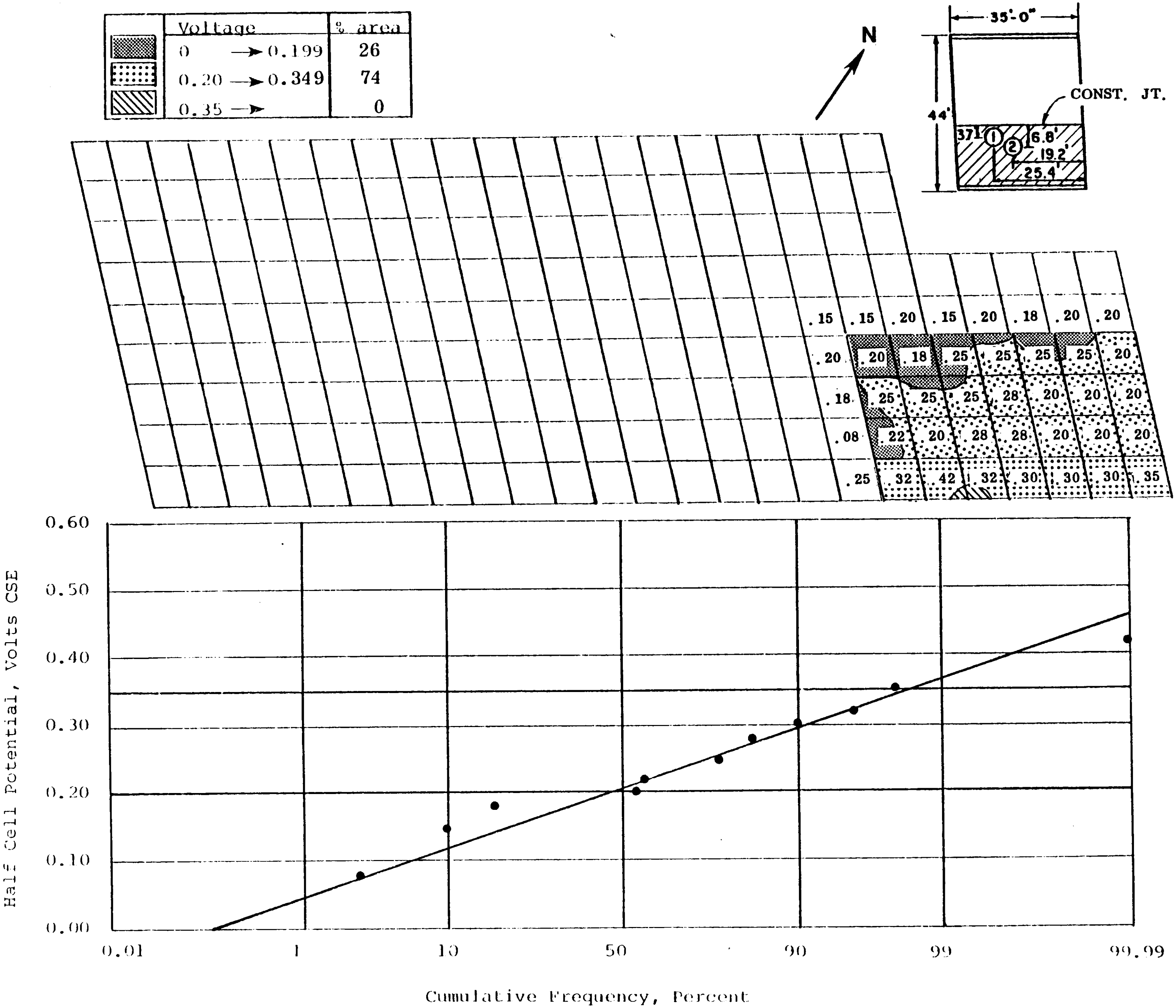


Figure 17. Corrosion potentials for bridge #6. All values in V-CSE.

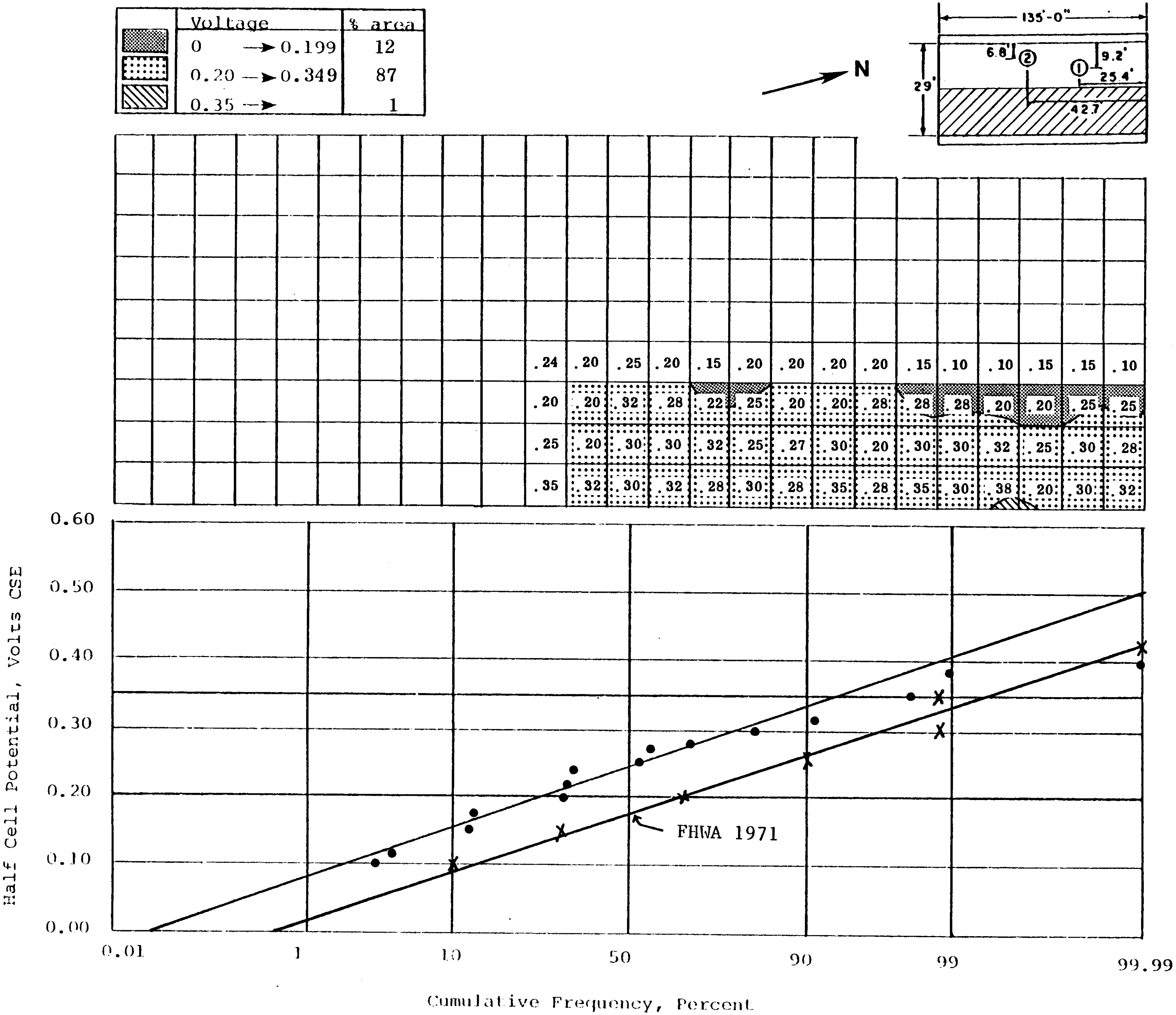


Figure 18. Corrosion potentials for bridge #7. All values in V-CSE.

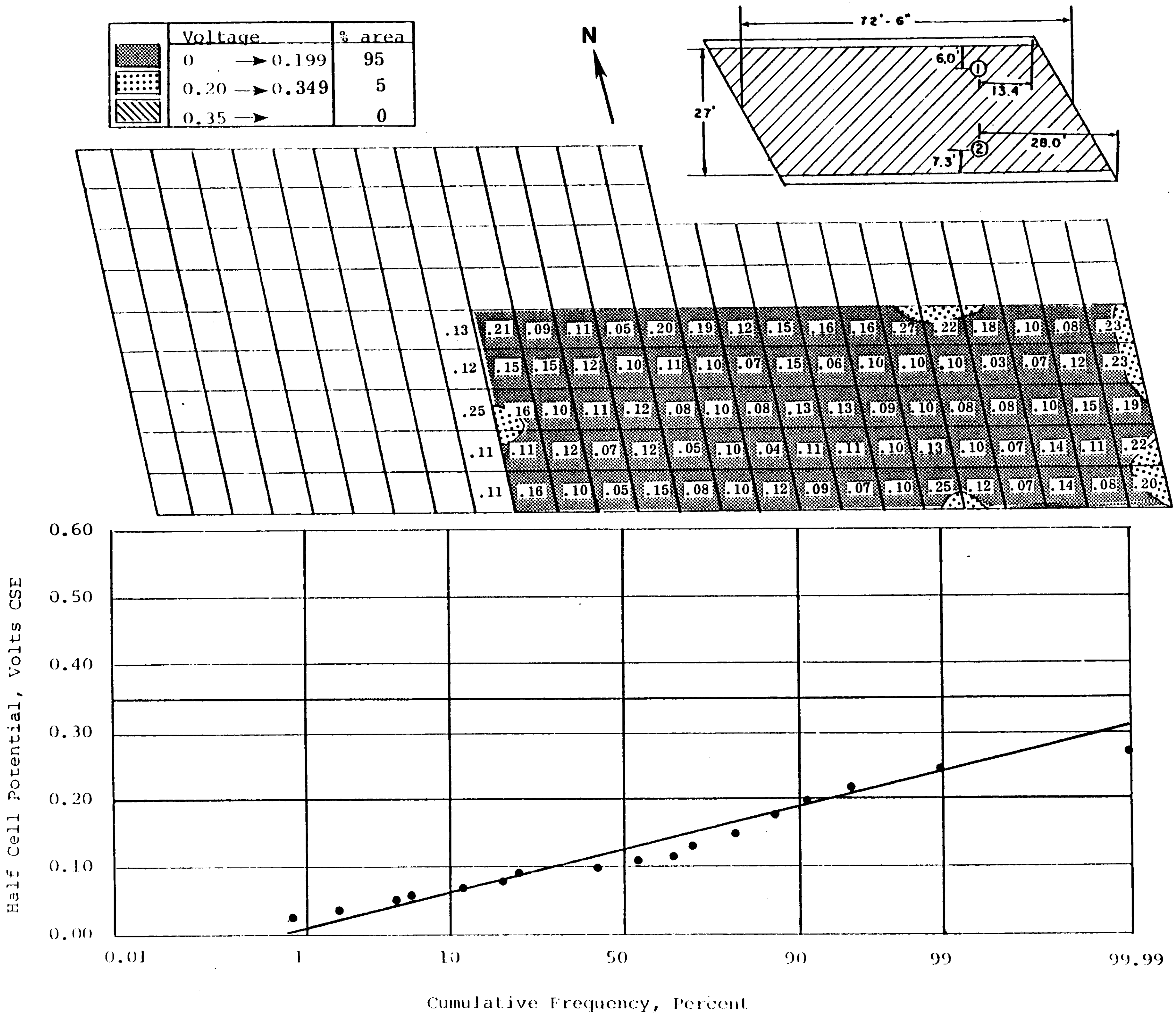


Figure 19. Corrosion potentials for bridge #8. All values in V-CSE.

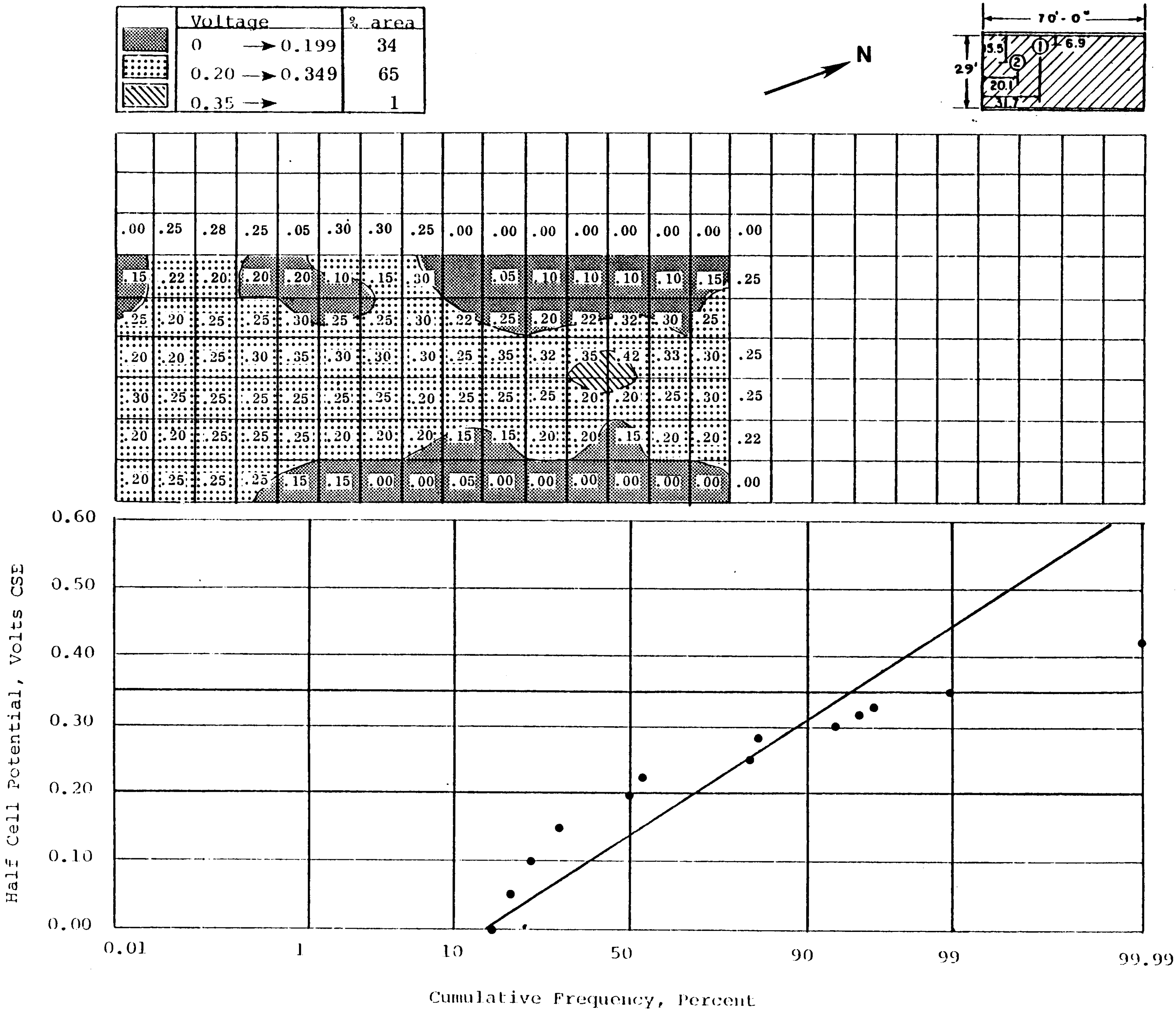


Figure 20. Corrosion potentials for bridge #9. All values in V-CSE.

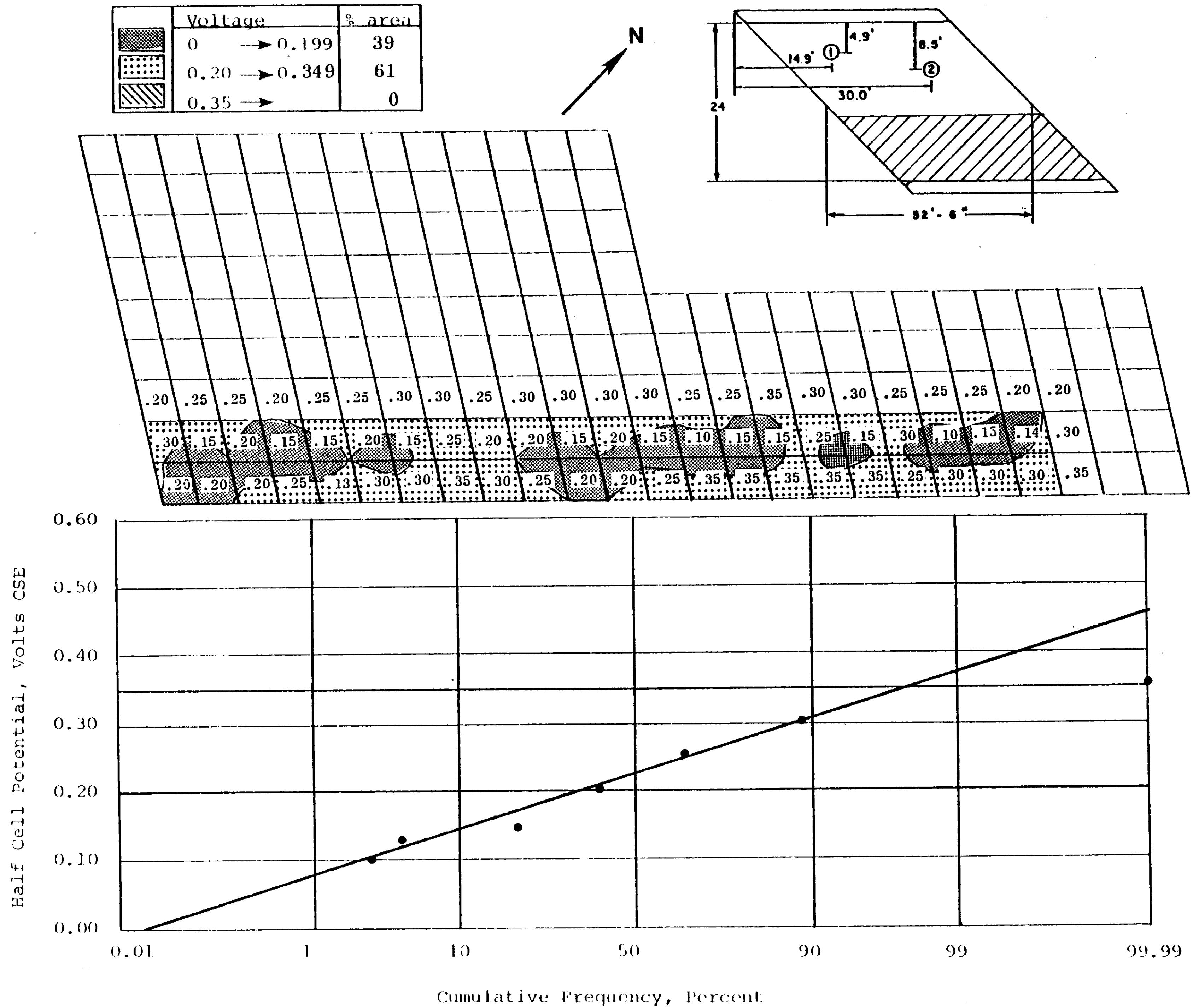


Figure 21. Corrosion potentials for bridge #10. All values in V-CSE.

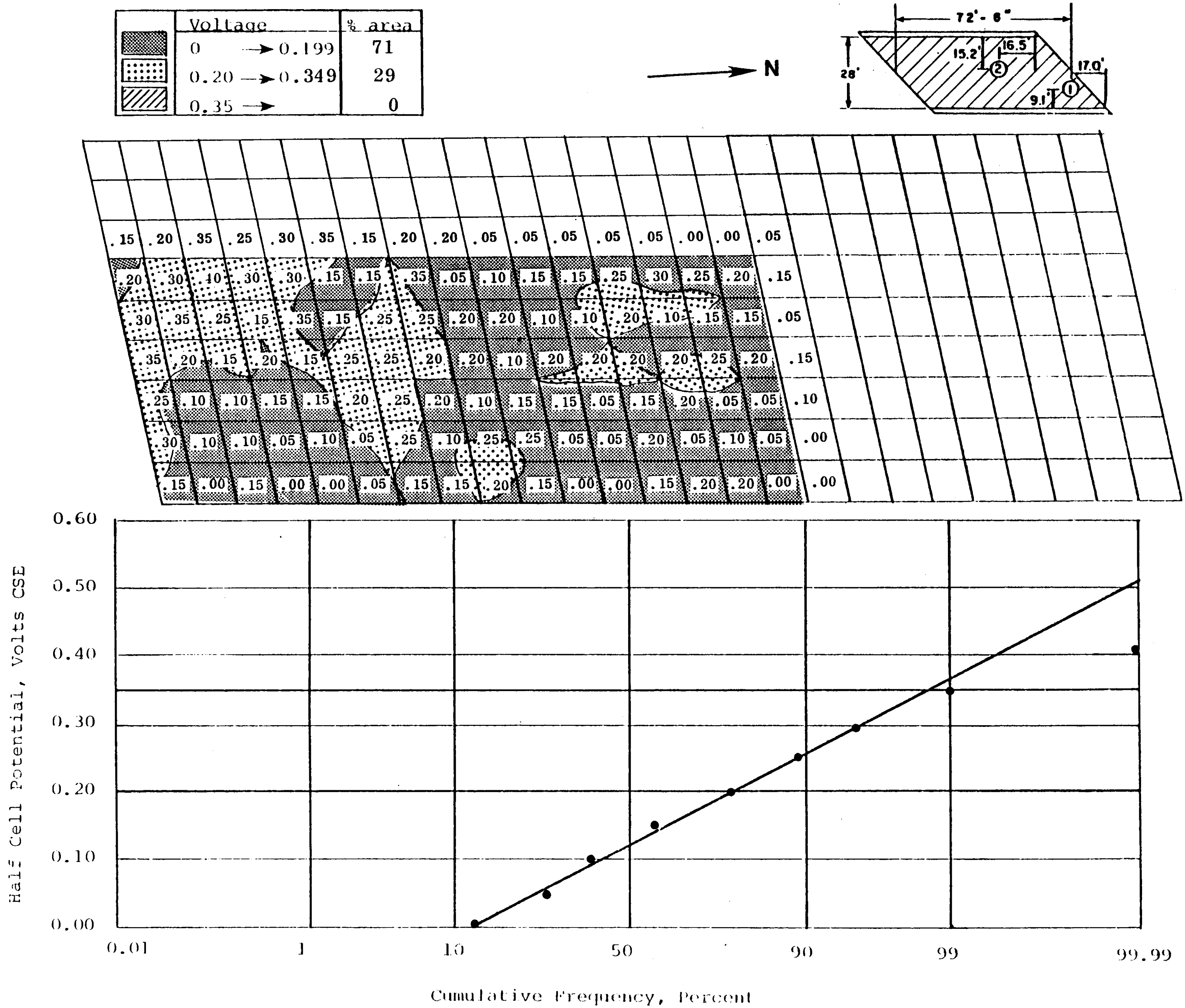


Figure 22. Corrosion potentials for bridge #11. All values in V-CSE.

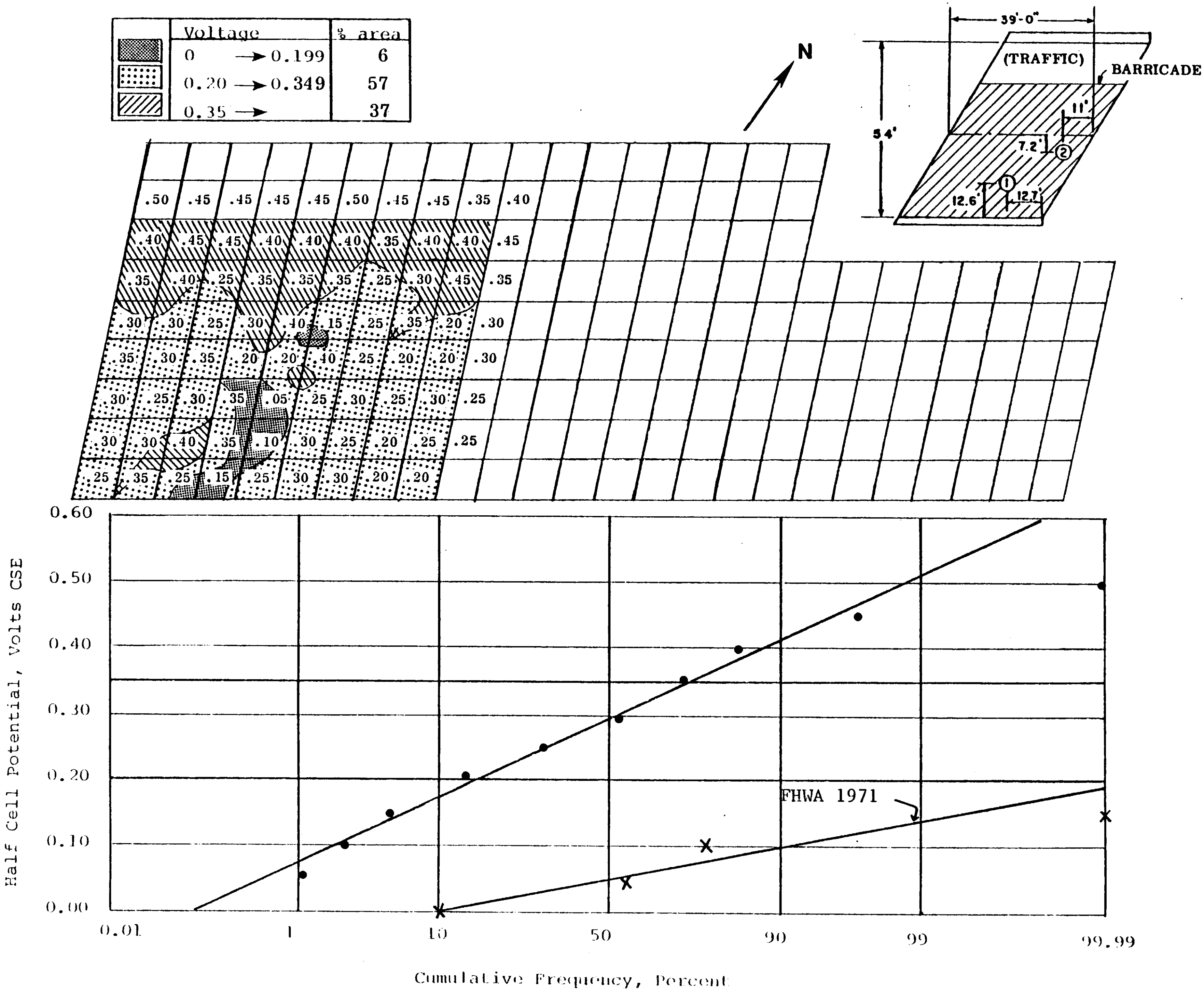


Figure 23. Corrosion potentials for bridge #12. All values in V-CSE.

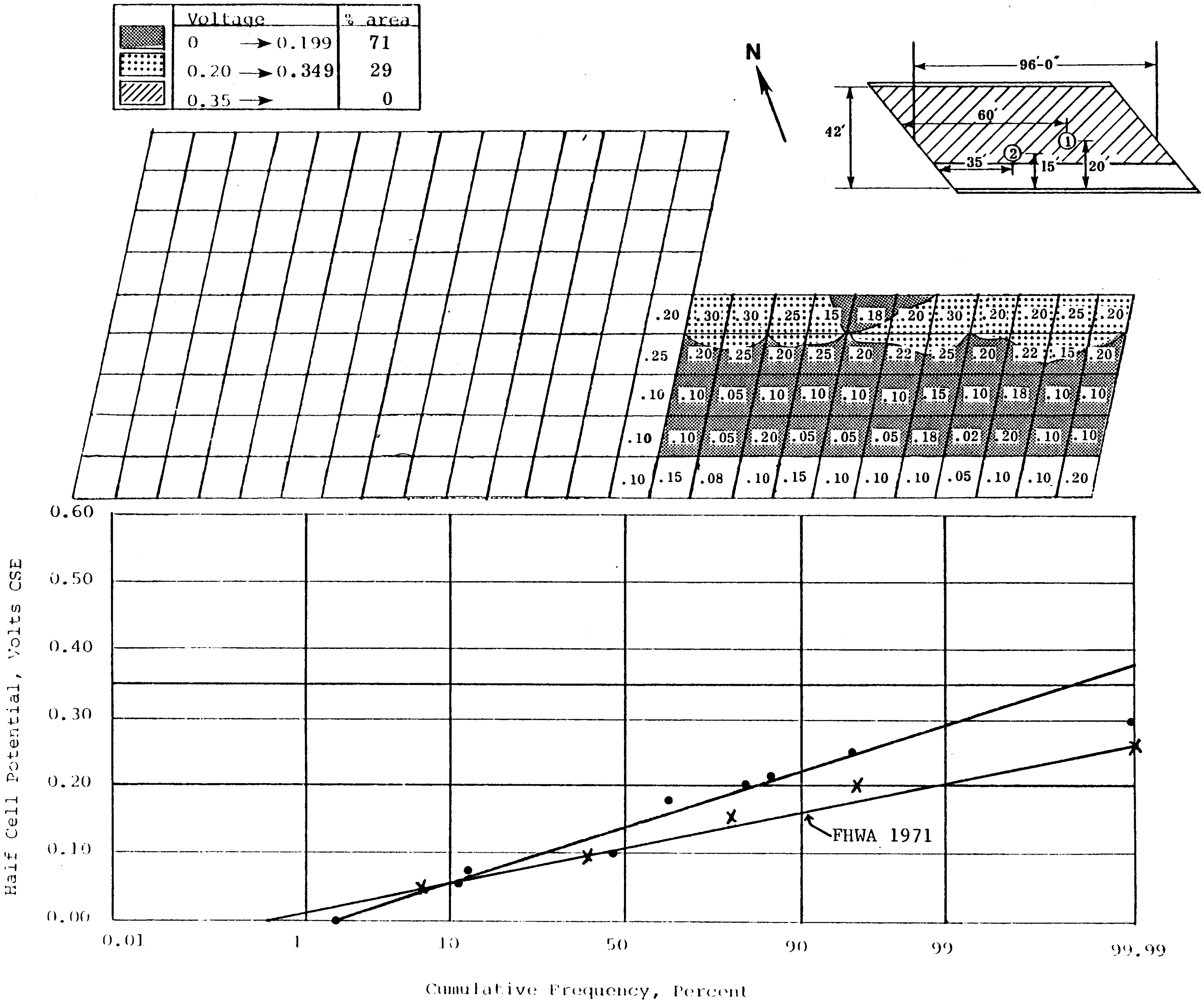


Figure 24. Corrosion potentials for bridge #13. All values in V-CSE.

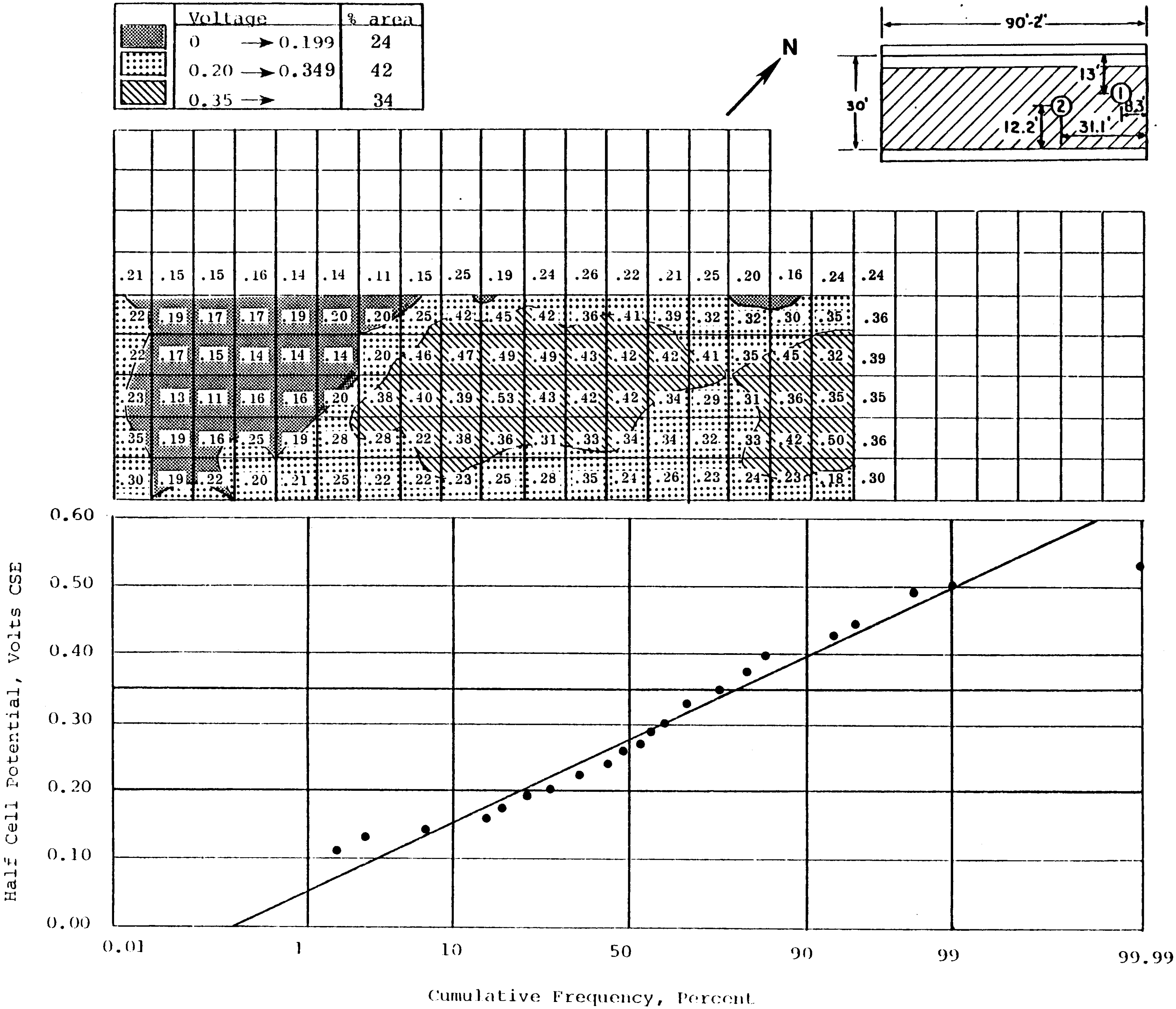


Figure 25. Corrosion potentials for bridge #14. All values in V-CSE.

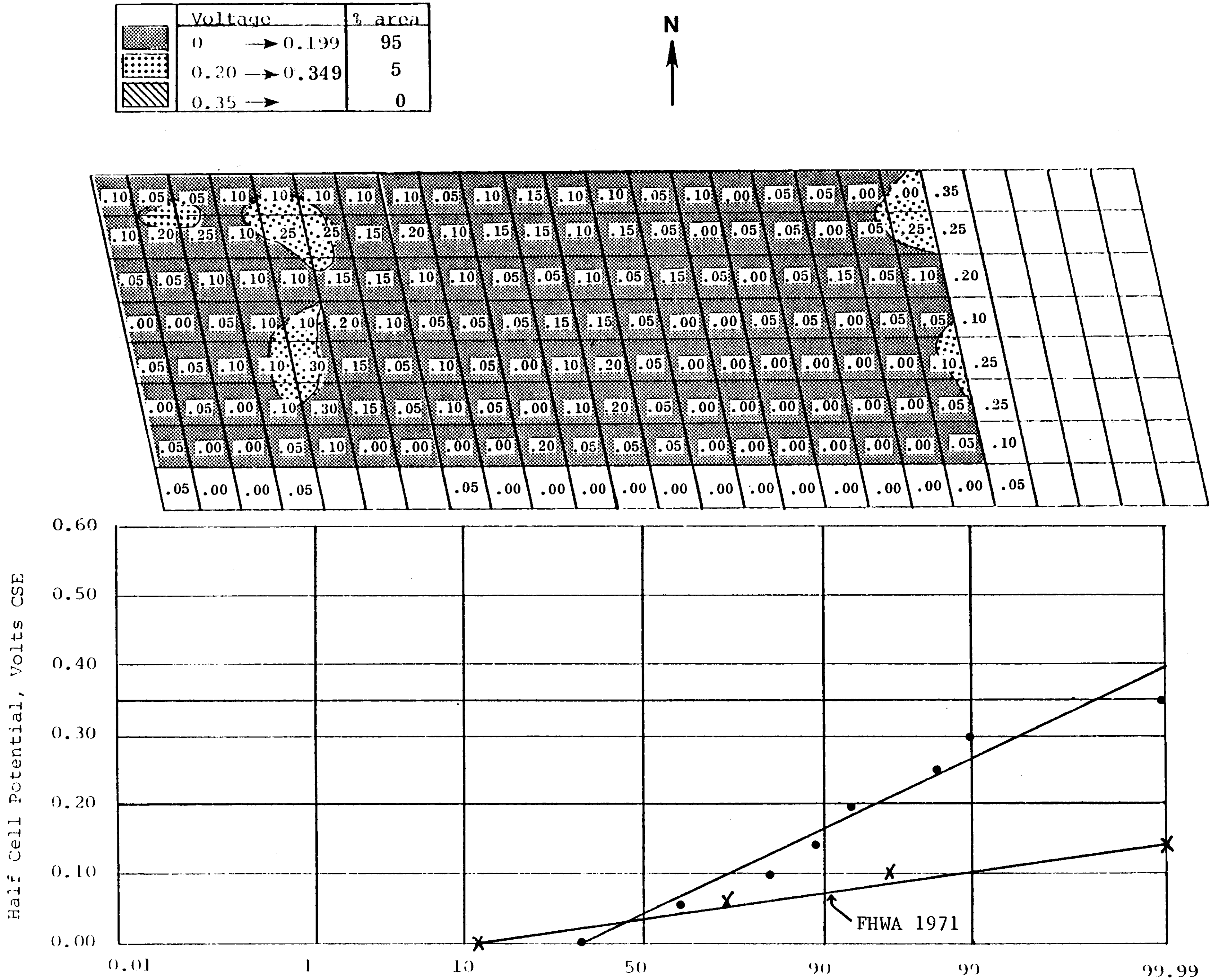


Figure 26. Corrosion potentials for bridge #15. All values in V-CSE.

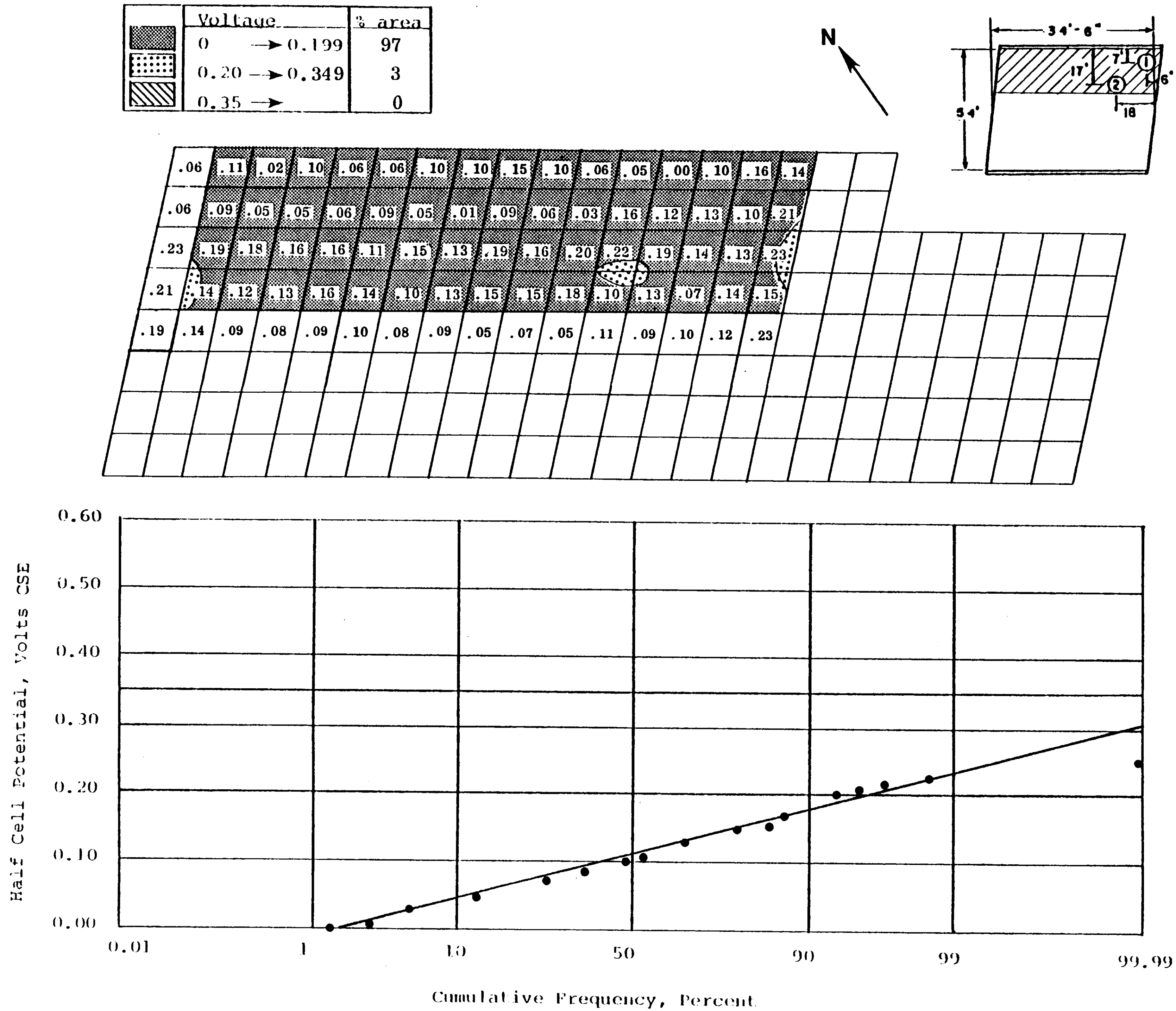


Figure 27. Corrosion potentials for bridge #16. All values in V-CSE.

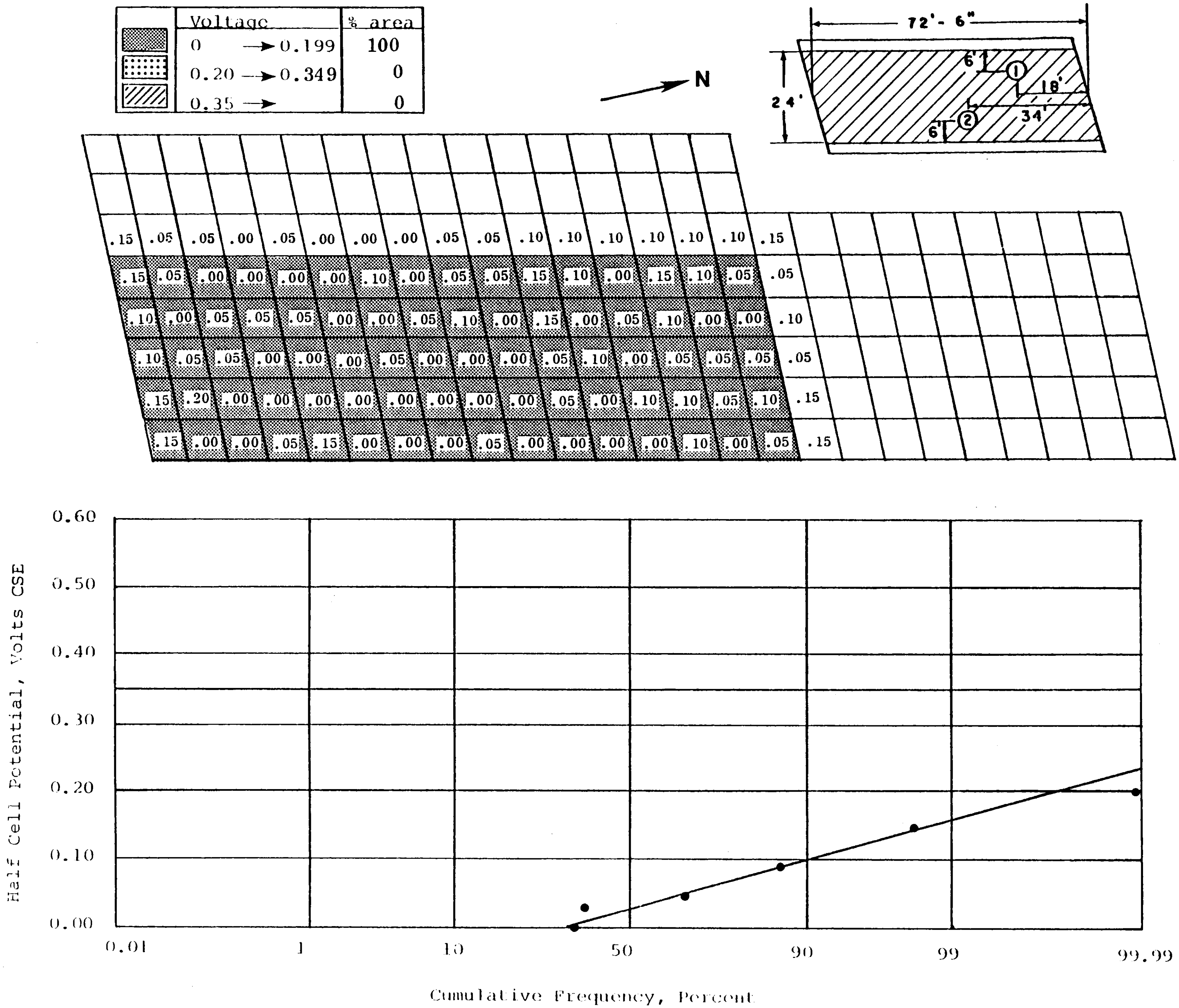


Figure 28. Corrosion potentials for bridge #17. All values in V-CSE.

While the cumulative frequency diagrams provide an indication of the general corrosion potential of the decks, the variability among decks and within a given deck as indicated by the areal diagrams in the figures is of interest. On all bridges but #1 and #17 there are "islands" of varying size with potential readings different from the predominate values most characteristic of the decks. The overlaid decks (#2, #4, #9, #11, and #14) are discussed later in the report. The isolated areas of highest potential are often associated with joints or edges of decks (bridges #3, #5, #6, #7, #8, #13, #15, and #16). These probably represent areas of high saturation encouraging accelerated ingress of chlorides. On bridges #5 and #6, the isolated areas of high potential coincide with areas where during construction situations were noted that could account for small areas of concrete that had poorer properties than those of the samples tested.

For example, on bridge #5, the area of high potentials in the southeastern corner is in an area in the "better" portion of the deck, as mentioned earlier in this report and discussed in more detail later. Construction operations on the eastern third of the deck provided concrete that was more satisfactory than that on the western two-thirds. A delay resulted in the embedment of the screed rail in the set concrete, and when work resumed the screed rail had to be dug out of the partially set concrete. The area was filled with concrete from the western end. This probably accounts for the isolated area of high potentials in the eastern corner. The areas where cold joints formed during the delay and a subsequent rainstorm also coincide in general with areas of higher potentials than adjacent areas. Although the diagram in Figure 16 represents only one-half of the deck, it might be speculated that the northern half would indicate a similar pattern.

On bridge #6 there is a small area of high potentials about 25 ft (7.6m) from the northeastern end. On this relatively small bridge all of the three loads of concrete required were placed very quickly, but because the quantities were underestimated a significant delay occurred in securing a partial load to complete the placement, and this resulted in a cold joint at the location of the high potentials.

The potentials measured on bridge #12 are of particular interest, since the major portion of the deck had not received traffic nor a direct application of deicing chemicals, but the portion north of the barricade had been heavily travelled and salted. It can be seen that areas of active corrosion are indicated adjacent to the barricade. The corrosion potentials are generally lower away from the barricade but there are isolated areas with high potentials. The corrosion potentials on this bridge reflect a condition that has been discussed in connection with repair procedures involving the removal of chloride contaminated concrete. Because corrosion is an electrochemical process, the conditions for corrosion are met if one end of a reinforcing bar is surrounded by chloride contaminated concrete while the other end is not.

Areas of active corrosion can occur outside the chloride contaminated area. This appears to be the case for bridge #12.

The area of higher potentials on the northern half of bridge #13 coincided with the lower side of the superelevated deck, which is consistent with the high degree of saturation that would be expected there, as was shown in Figure 8.

In general, the bridges that showed the best resistance to the penetration of chlorides as indicated in Table 6 (bridges #1, #15, and #16) had the lowest corrosion potentials, as would be expected.

As noted, the data gathered in 1977 were the first systematic measurements of corrosion potential made on the 17 decks. During the FHWA's development of the method, however, measurements were made as part of Demonstration Project No.15 on 4 of the decks in 1971 (bridges #7, #12, #13, and #15). The FHWA data are included in the cummulative frequency portions of the relevant figures. A comparison of the data gathered at these two times, 6 years apart and by different agencies, is of interest. The areal diagrams prepared from the 1971 FHWA measurements for bridges #7 and #13 are shown in Figures 29 and 30 and compared with the 1977 data.

The 1971 surveys indicated no areas with potential readings greater than -0.26 V CSE on bridges #12 and #15. Except for bridge #12, there was only a modest increase in corrosion activity between 1971 and 1977. The 1971 data for bridge #12 indicated that the entire deck had a potential below -0.20 V CSE, whereas by 1977 the situation had deteriorated as shown in Figure 23. The reason for the large change in bridge #12 has been previously discussed.

The agreement between the two separated measurements on the remaining 3 bridges was surprisingly good. The progression of corrosion on bridges #7 and #13 was small. While that on bridge #15 was larger, it occurred in isolated areas. There were no areas on bridge #15 greater than -0.20 V CSE in 1971. The comparative areal diagrams for bridges #7 and #13 shown in Figures 29 and 30 tend to show that areas of high potential in 1971 remained high in 1977. The agreement for bridge #13 was particularly good.

The interpretation of corrosion potential measurements is complicated by the fact that the method of measurement does not precisely locate the corrosion but rather indicates whether or not there is a potential for corrosion in the deck. It measures the transfer of electrons between the anodic and cathodic areas that develop along the reinforcing bar. Because the distance between an anode and a cathode may range from less than 1 in (25 mm) to more than 20 ft (6 m), the difficulty of precisely locating active corrosion is obvious.

	Voltage
	0 → 0.199
	0.20 → 0.349
	0.35 →

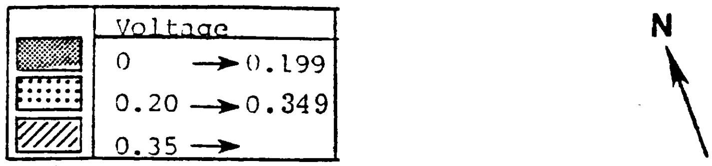
.08	.19	.14	.06	.17	.12	.20	.24	.06	.12	.15	.16	.13	.16	.12
.18	.18	.13	.21	.16	.21	.17	.11	.16	.15	.22	.19	.14	.20	.23
.08	.20	.16	.20	.17	.13	.22	.10	.14	.23	.12	.17	.26	.10	.17
.25	.26	.21	.23	.11	.25	.42	.25	.19	.26	.25	.23	.30	.23	.28

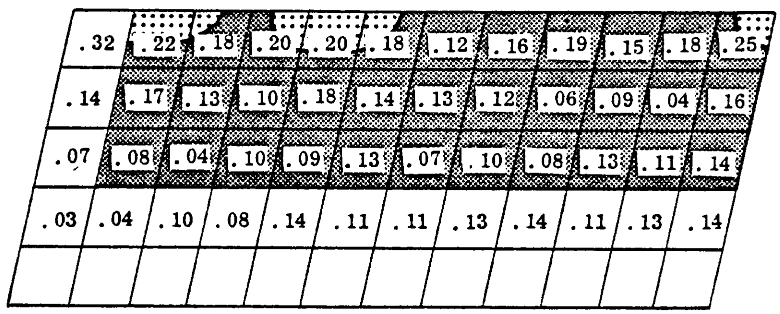
(a) 1971 FHWA Data

.24	.20	.25	.20	.15	.20	.20	.20	.20	.15	.10	.10	.15	.15	.10
.20	.20	.32	.28	.22	.25	.20	.20	.28	.28	.28	.20	.20	.25	.25
.25	.20	.30	.30	.32	.25	.27	.30	.20	.30	.30	.32	.25	.30	.28
.35	.32	.30	.32	.28	.30	.28	.35	.28	.35	.30	.38	.20	.30	.32

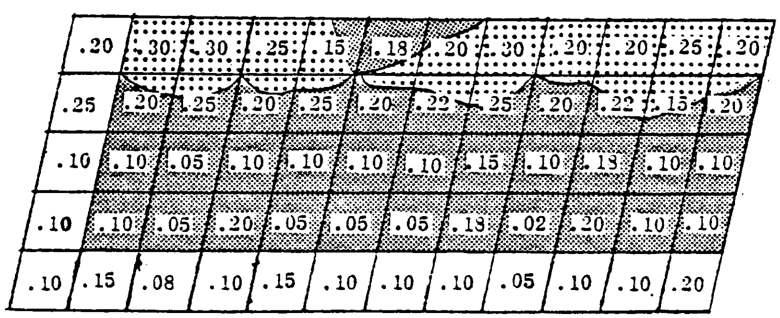
(b) 1977 VHTRC Data

Figure 29. Corrosion potentials for bridge #7 measured in 1971 and 1977.





(a) 1971 FHWA Data



(b) 1977 VHTRC Data

Figure 30. Corrosion potentials for bridge #13 measured in 1971 and 1977.

The ingredients necessary for corrosion are moisture, oxygen, and chlorides. If there is sufficient chloride to destroy the passivity of the reinforcing steel, then corrosion will continue, assuming that moisture and oxygen are present. Considerable research has been directed toward establishing the threshold value of chlorides necessary to initiate corrosion. For a typical bridge deck concrete, the value conventionally used is 0.20% of total chlorides by weight of concrete, which converts to 0.033% Cl^- by weight of concrete.

Because of the uncertainty of the location of the corrosion activity and the variability of chlorides in a deck, only general relationships can be established. It is generally agreed that at least six chloride samples are needed to characterize a deck. Since in this study only two were taken, it is not possible to define the relationship between chloride content and corrosion activity. Nevertheless, some trends are evident in the data given in Table 9. In Table 9 the decks are listed in decreasing order of corrosion activity, along with the chloride ion concentration at the 1 3/4-in (44-mm) level. The behavior of the five overlaid decks will be subsequently treated. The Cl^- levels below the threshold value of 0.033% are indicated by parentheses.

For the five bridges with the least corrosion activity (#1, #8, #15, #16, and #17) both core samples contained less than the threshold value. Neither sample from bridge #12 contained Cl^-. The high corrosion activity on this bridge as a result of the heavily contaminated adjacent deck has been discussed. Of the remaining six uncovered decks at least one sample from each contained greater than the threshold level of Cl^-.

Table 9

Chloride Ion Contents and Corrosion Potential Data

Bridge	Cl^- at 1 3/4 in (44mm) Sample 1	Sample 2	Area of Deck Having Shown Potential >-0.35V	<-0.35>0.20V	<-0.20V
2[a]	0.059	(.008)	43	28	29
12	(0)	(0)	37	57	6
14[a]	Overlaid before traffic		34	42	24
5	(0)	0.123	13	46	41
4[a]	(0.021)	0.084	2	62	36
7[b]	0.013)	(0.027)	1	84	12
9[a]	No Sample	(0.027)	1	64	34
3	0.36	.113	1	35	64
6[b]	(.004)	(0)	0	74	26
10[c]	.040	(.022)	0	61	39
11[a]	.037	.053	0	29	71
13	.058	(.021)	0	29	71
8	(.029)	(.023)	0	5	95
15	(.002)	(0)	0	5	95
16	(0)	(0)	0	3	97
1	(.001)	(.003)	0	0	100
17[c]	(0)	(0)	0	0	100

a Overlaid
b Silicone Surface Treatment
c Minimal deicing
( ) Acceptable Values

Both samples from bridges #6 and #7 had less than threshold levels of Cl^- but greater than average corrosion potentials. This is consistent with the observations of scaling on these bridges previously discussed. These decks received silicone treatments, which have been shown to raise the moisture content of concrete and thus would account for the high corrosion potentials as well as the increased tendency for scaling.

In summary, the results of the corrosion potential measurements on the uncovered decks are in excellent agreement with other findings in this study. High corrosion potential measurements do not mean that extensive corrosion is occurring but that the proper conditions for such corrosion exist.

Measurements of corrosion potentials have been criticized as being variable and difficult to interpret. The results from this study illustrate why this is so if these measurements or chloride contents are used alone. For example, for bridges #6, #7, and #12 the corrosion potentials would indicate a possible cause for concern while the chloride data would not.

Based upon these data, one might suggest that high potentials and high levels of Cl^- should be of concern. If only one of these parameters is high, conditions of moisture saturation or other site specific factors must be evaluated.

Rusting and Spalling

Rusting of the reinforcement and subsequent deck spalling are primarily functions of the depth of concrete cover, and as the cover depth increases, of the quality of the concrete. Visual evidence of rusting or surface spalling is a clear indication of corrosion as opposed to the potential for corrosion indicated by electrical measurements or chloride determinations.

Cover depths were not measured in the field but 29 of the 34 cores removed intersected reinforcing bars. The depths to the top of the uppermost embedded bars (clear cover) on these cores were measured, and the average clear cover was found to be 2.34 in (59mm) with a standard deviation of 0.495 in (12.6mm). The extreme values were 1.75 in (44mm) and 3.25 in (82mm). These values are consistent with the results of an earlier study in which clear cover depths were determined with a pacometer for 68 bridges (Newlon 1974). That study included 6,310 measurements on which the specified clear cover was 1.69 in (43 mm). The average clear cover obtained from this larger sample was 2.34 in (61 mm) with a standard deviation of 0.47 in (12 mm), a remarkable agreement with the values obtained from the current sample.

As discussed in earlier reports (Davis, North, and Newlon 1971; Newlon 1974), the fact that the cover specified by the Virginia Department of Highways and Transportation has always been comparatively large, 1.69 in (43 mm), and has usually been exceeded in practice, is believed to account for the relatively infrequent occurrence of spalling in Virginia decks. In 1966 the required clear cover was increased to 1.94 in (49 mm), and in 1974 to 2.00 in (50mm).

In the current study no correlation was found between the comparatively few spalls observed and the average cover depths. It is probable that the spalls were associated with isolated bars that happened to be misplaced.

No rusting was observed on any decks surveyed. It should be noted that some spalling that had developed in several of the decks when they were overlaid because of insufficient cover would not be reflected in the 1977 survey.

The absence of rusting or spalling does not signify the absence of a potential for corrosion, as previously discussed.

Behavior of Overlaid Decks

Of the 17 decks originally in the study, 5 had been overlaid before the 1977 survey. Of these, bridges #4, #9, and #14 had been constructed using a longitudinal screed that resulted in general or localized deficiencies in cover over the reinforcement, accompanied in some cases by other deficiencies in the properties of the concrete (Hilton 1971). The deficient cover on bridge #4 was highly localized and occurred over bars bent upward near the ends of the span. Bridge #14 was overlaid prior to receiving any traffic to correct deficient cover resulting from an incorrect deck thickness. Bridge #9 has had multiple deck repairs, largely because of deficient cover but also because of scaling and general deterioration of the concrete.

Bridge #11 was overlaid to correct surface wear and scaling explainable in terms of the usually poor curing previously described. Bridge #2 was overlaid to increase low skid resistance resulting from use of a polish-susceptible fine aggregate. Bridge #5 was overlaid in 1977, soon after the condition survey, to correct the numerous problems previously discussed.

Because these overlays were not studied as part of this project, no extensive discussion of these decks is possible. Only the data obtained from chloride analyses and corrosion potential measurements are included. The basic characteristics of the overlays are summarized in Table 10.

Since bridge #14 has been covered with a bonded portland cement concrete overlay for its entire service life, with respect to chloride penetration it has performed as would a monolithic deck, as illustrated in Figure 31. The reinforcing steel was at the top of the original deck, and thus at the bottom of the overlay. As can be seen, the chloride distributions in the overlay and in the deck were similar to those shown earlier for the uncovered decks. The chloride levels in the overlay were as high as any found in the study, although the highest value of 0.272 approximates the value of 0.236 found in sample 16-2. Bridge #16 is close to and on the same route as bridge #14. It also appears that the epoxy bonding compound had significantly reduced the penetration of chlorides, as evidenced by the chloride content immediately above and below the bond line. Research by the Council has demonstrated that a bonding grout is unnecessary for structural purposes in two-course deck construction (Tyson 1976), but these data indicate that the epoxy material used in this deck was effective in reducing chloride penetration. Such a reduction would not be expected if a portland cement grout were used.

Table 10

Characteristics of Overlaid Decks

Bridge No.	Age When Overlaid, Years	Defect	Type of Overlay
2	7	skid resistance	seal -- with coal tar epoxy -- 1½ in (33 mm) bituminous concrete
4	8	localized deficient cover	seal -- with coal tar epoxy -- 1½ in (33 mm) bituminous concrete
9	a. 6.	deficient cover and scaling	epoxy mortar
	b. 9.	failure of overlay	1 1/2 in (33 mm) bituminous concrete
	c.14	various	deck restoration, patching, overlay with 3/4 in (19 mm) latex concrete
11	12	surface wear	epoxy seal plus silica sand
14	0	deficient deck thickness and cover	2 in (50 mm) portland cement concrete bonded with epoxy

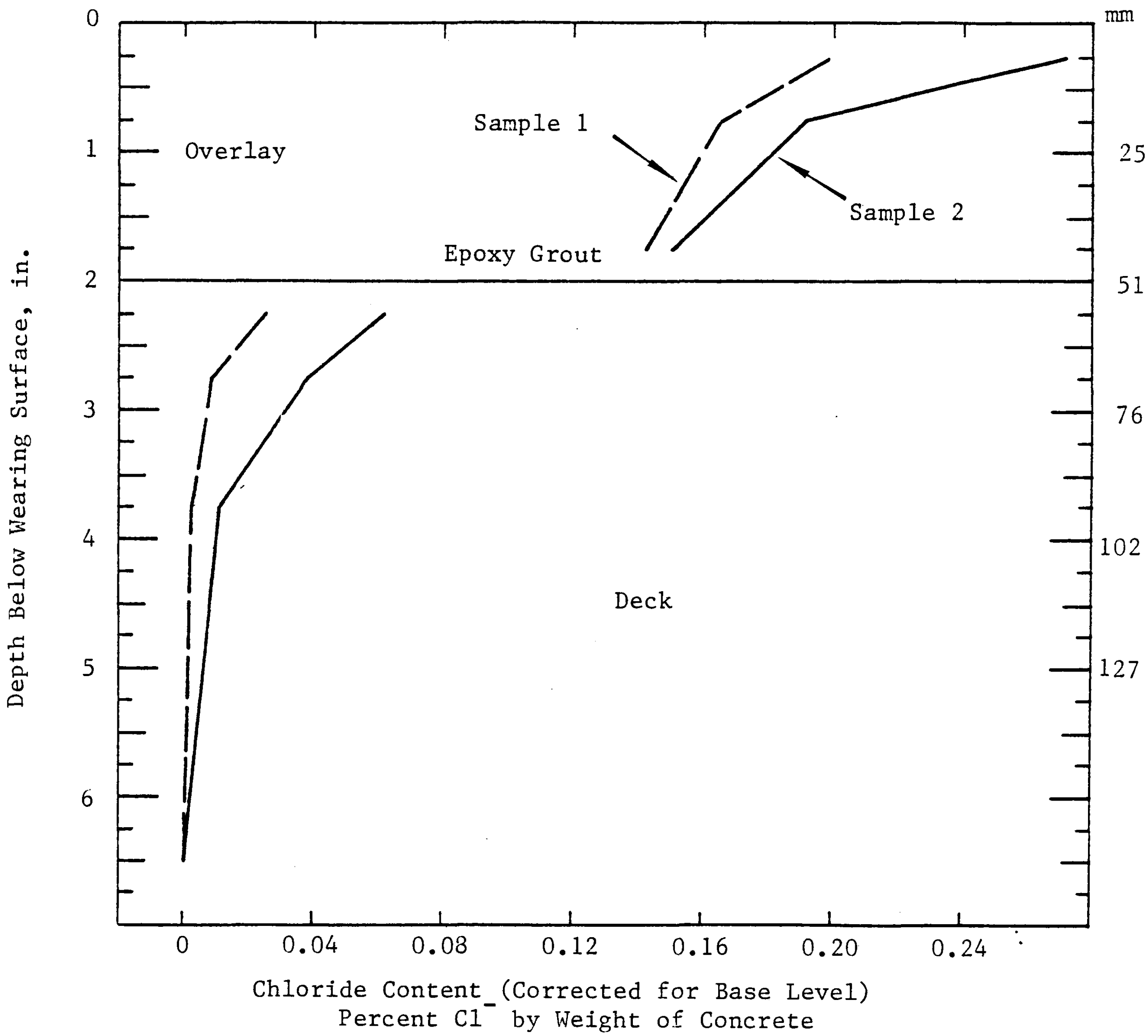


Figure 31. Chloride distribution in bridge #14, which was overlaid prior to traffic.

The corrected chloride contents for the overlaid decks are shown in Table 11. These values are quite high, with the exception of samples 2-2, 4-1, and 11-1 at the 1 3/4-in (44-mm) level. The low values in Table 11 for samples from bridge #14 do not reflect the higher values in the overlay previously discussed.

The values for bridge #9 are high and variable, thus reflecting the extensive repairs. The values given are uncorrected, but since the coarse aggregate was a granite, they should be reduced about 0.006% to reflect the chloride in the aggregate based upon previously cited studies (Clemena and Reynolds 1980). It is significant to note the very high absorption values for bridge #9 and the correlation between the relative amounts of chloride and the differences in absorption on a given bridge. As noted earlier, bridges #2 and #4 were overlaid for reasons other than concrete deterioration, a situation consistent with the low absorption values.

There is considerable controversy as to the ability of overlays to protect concrete from chloride penetration and, in fact, increased chloride contents have sometimes been encountered after overlays have been placed. Since no information on chloride content prior to resurfacing is available, no comparison can be made. Regardless of whether or not the chlorides were high before application of the overlay, the overlaid concrete contains chloride in levels ample to initiate corrosion beneath the overlay.

Figures 13, 15, 19, 21 and 25 show significant areas of active corrosion, particularly for bridges #2 and #14. Half cell measurements for overlaid decks must be interpreted with caution, since the moisture in or below the overlay can conduct sufficientions for large distances and not necessarily along a bar. Thus, the locating of corrosion sites is difficult, but the half cell values combined with chloride contents of the magnitude found in the samples strongly suggest that corrosion was in progress.

Table 11

Chloride Ion Concentrations for Concrete in Overlaid Decks

		Chloride Content, % Cl^- by Weight of Concrete (corrected for base level)			
		¼-½ in	3/4 in	1 3/4 in	Absorption,
Sample	Overlay [a]	(6-13mm)	(19mm)	(44mm)	Percent
2-1	Epoxy Seal Plus Bituminous Concrete	0.122	0.095	0.059	4.65
2-2	"	0.052	0.032	0.008	4.50
4-1	"	0.093	0.067	0.021	4.75
4-2	"	0.146	0.109	0.084	5.03
11-1	Epoxy	0.111	0.054	0.037	5.01
11-2	"	0.098	0.130	0.057	6.37
14-1	2 in (50mm) P.C. Concrete Bonded With Epoxy [b]	0.024	0.008	0.001	5.08
14-2	"	0.065	0.028	0.009	5.25
9-1 [c]	Various	0.062	0.125	No Sample	5.81
9-2 [c]	"	0.143	0.191	0.093	6.84
				0.066 at 4 1/2 in (114mm)	

a See Table 10

b See Figure 31 (The depth below wearing surface is 2 in (50mm) greater than indicated.)

c Uncorrected values

Cracking

Cracking of the decks, as indicated in Table 4, was comparatively limited, with the possible exception of that on bridges #6, #7, and #13. It is of interest to note that the concrete used in bridges #6 and #7, as well as that in #1, contained fine aggregate from a source that during the period of this construction contained more clay than was desirable. As noted in Newlon's 1971 report, excessive cracking of concrete pavement constructed concurrently with the bridges (Newlon 1965) indicated that the pavement cracking was thermal or "morning" cracking compounded by the excessive clay content that increased drying shrinkage (Brown 1965). Additional washing of the sand reduced cracking in the pavements constructed subsequent to the placement of the decks on bridges #1, #6, and #7. There is strong evidence that the poor quality of the fine aggregate contributed to the shrinkage cracking on these decks. Similar supporting information is not available for the materials used in bridge #13. The source used in bridges #12 and #13 were from the same source (different from those in #1, #6, and #7). No relationships between cracking and traffic volumes or structure types were found. It should be noted that none of the decks were continuous spans, except for the massive slab of bridge #10.

In summary, the only cracking of any significance appeared to have been caused by the use of sand with a lower than desirable sand equivalence value.

Popouts

A few popouts were observed on bridges #3, #6, and #15. In these decks, except for that on bridge #3, the coarse aggregate was a siliceous gravel containing a minimal amount of chert. It is probable that the defects classified as popouts on bridge #3 were not classic popouts since the coarse aggregate was a granite that is not known to cause popouts. Extensive popouts were observed on bridges #12 and #13, where the coarse aggregate used was a siliceous gravel with significant amounts of low specific gravity, altered chert susceptible to damage by freezing and thawing as demonstrated in a study by Newlon, Ozol, and McGhee (1965). The percentages by volume of altered chert observed in the four cores from the two structures varied between 14.7% and 22.2%, which are amounts sufficient to cause extensive popouts.

The Special Case of Bridges #4 and #5

As noted in earlier discussions, bridge #5 involved special circumstances that developed during construction. While the conditions under which bridge #5 was placed were unusual in the context of the present study, this case dramatically demonstrates the interacting and snowballing effects of the numerous operations that must be coordinated during deck placement. It also helps to explain performance that in the absence of such knowledge would be unexplainable.

Bridges #4 and #5 were placed simultaneously by different contractors at different locations but using concrete from the same ready mix plant. Because of the simultaneous construction, it is best to let the summary reports prepared immediately after construction speak for themselves concerning the effect on bridge #5.

General Comments Concerning Bridge Jobs No. 4 and 5

The general conduct of concreting on both of the above jobs was extremely poor as evidenced by the fact that it took almost 12 hours to complete the 84-foot span on job No. 5. The following general observations apply to both jobs and are intended to supplement the detailed information given for each job.

As a result of an earlier record sample which had shown a slump greater than 5 inches, there was a considerable effort to exercise very close concrete control on these jobs. This was evidenced by the fact that on Job No. 5 the inspector ran more than 40 slump tests and 35 Chace air tests. A proportionate number of tests were made on Job No. 4.

The reason for the above record failure was said to be that water was added to the last portion of a truck between the inspector's tests and the taking of the record samples. In order to remedy this condition it was decreed that all water would be added at the plant and that none would be added on the job.

There was apparently a considerable slump loss in some cases and the concrete on job No. 4 was relatively stiff. The slump on job No. 5 was generally higher but the concrete stiffened rather soon after placing.

It appeared that a considerable amount of the trouble on these jobs was caused by an inaccurate moisture determination at the plant. The only difference in the concrete used on the two jobs was that the sand on job No. 4 was a natural one, whereas that on job No. 5 was manufactured. Generally speaking, the concrete on job No. 4 was too stiff and the concrete on job No. 5 was about as wet as would be desirable.

The primary controlling feature on the above jobs was the inability of the ready-mix producer, who was supplying both jobs simultaneously, to supply concrete at a constant rate. Delays ranging from 15 minutes to almost 1 hour were encountered on job No. 5 and lesser delays on job No. 4.

Memorandum Report Job No. 5

(1) General Concreting Operation. Placement began at the east end of the slab at approximately 8:00 a.m. and proceeded westward. Due to the delays and other circumstances which have been discussed in the general comments, the concreting operations were broken into approximately 3 parts: the first consisted of placement in the eastern third of the slab, the second in the central portion of the slab, and the third in the western 25 feet of the slab. Concreting on the first section was completed at approximately 10:00. Concreting in the second section proceeded from about 11:30 to 2:30. Concreting in the third section proceeded from about 4:30 until 6:00. The order of operations was as follows:

(a) Placement with a crane and bucket
(b) Spreading and internal vibration
(c) Several passes with a transverse oscillating screed
(d) Smoothing with a "bull" float
(e) Dragging with a wet burlap drag
(f) Brooming
(g) Curing with curing compound

(2) Atmospheric conditions. High overcast sky with strong breezes across the span area at 8:00 with the sun breaking through the clouds at 10:00 followed by increasing clearing. A severe thunderstorm with considerable wind and rain occurred at 4:00 between concreting in areas 2 and 3. This storm was followed by clearing and conditions very similar to those which prevailed prior to the storm.

(3) Concrete. The concrete was supplied by the local ready-mix company (same as Job #4). As noted previously there was a considerable problem in maintaining a steady flow of concrete and in securing a consistent workability. In the beginning no water was added at the job. Throughout the day the concrete had a relatively high slump, but stiffened rather early so some water was added on the job. Concrete was placed from 6 trucks during the period from 8:00 a.m. to 9:52. Trucks 7, 8 and 9 were rejected with excessive slumps of 5 inches, 7 inches, and 7 inches respectively. The relatively long gap between trucks plus these rejections resulted in no concrete arriving on the job between 9:52 and about 11:30. The result was a cold joint between areas 1 and 2. Area No. 1, the eastern third, was finished and cured before

any more concrete arrived. Concreting was resumed about noon and proceeded in area 2 until it reached an area approximately 25 feet from the western end of the slab. Sample 5-1 was taken in this area. At about 2:30 there was another considerable delay in trucks and no concrete arrived on the job from about 2:45 until approximately 3:45. During this period section 2 was finished and cured. At 3:50 there was a severe thunderstorm and the surface of area 2 was rather severely washed away. Because of the storm no concrete was placed from 3:55 until 4:30. At 4:30 concreting was resumed on the end 25 feet. Some concrete in this area which had been in place prior to the rain but had not been finished was severely washed by the rain and resulted in excessive coarse aggregate at the top. This concrete was turned over with shovels such as to give a mortar like appearance and the remaining concrete was placed in this area from 4:30 until about 5:45 or 6:00. Sample 5-2 came from this area.

(4) As on job No. 4 the finishing operation was severely handicapped by a lack of concrete. Because the transverse finisher operated off screed poles set in the slab, a problem arose when the delay between areas 1 and 2 occurred. The cold joint was formed at the middle of a screed pipe, and thus by the time the concrete was placed in the second area the screed rail had set up in the concrete placed in the first area. It was necessary to dig this concrete out with a pick. The concrete was sufficiently hard to require a considerable digging effort and a rather large trough approximately 3 inches deep by 1 foot wide by 15 to 20 feet long was dug out and replaced with a cement-sand mortar which was worked into the trough by hand. Because of the delays which have been discussed, this situation prevailed to a lesser degree in several places and on this bridge deck there were a number of cases in which it was necessary to patch the area from which the screed pipes were removed. Because the work bridge was attached to the mechanical screed, it was not feasible to utilize this work bridge when it was necessary to work the cold joint because the screed was some 30 to 40 feet in front of the place where the bridge was needed.

(5) Curing. The curing compound was spread with a long sprayer and extremely good coverage was obtained.

Summary. The same general comments given on job No. 4 are applicable to this job also, and in general the concreting operation was extremely poor.

Meanwhile, on Bridge #4, related difficulties were being encountered.

(1) General concreting operation. Concreting began with the placing of a median beginning at 8:00 a.m. and lasting until approximately 8:30. Concreting of the deck began in the northwest corner at 9:40 a.m. Concreting of the deck proceeded from the northern side to the southern side and was completed at approximately 12:30 p.m. The order of operations was as follows:

a. Placement by crane and bucket.

b. Internal vibration.

c. Several passes with a longitudinal oscillating screed.

d. Belting longitudinally with a 5 inch canvas belt.

e. Curing compound (hand sprayed).

The above procedures pertained to the entire deck with the exception of approximately 1 foot around the periphery that was hand floated. It is to be noted that there was no brooming on this deck.

(2) Atmospheric Conditions. The early portion of the morning was slightly overcast and changed to high sky with moderate winds of about 8 miles per hour. The atmospheric conditions contributed slightly to drying.

(3) Concrete supplied by the local ready-mix company (same as Job #5). All materials, including water, were added at the plant. The concrete was mixed for 70 revolutions at the plant and hauled at agitating speed to the job. For the reasons previously cited the concrete was rather stiff during the initial part of the pour. The mix design was based upon a 7% sand moisture. With the truck following that which delivered sample 2, the mix was adjusted on the basis of a 4% sand moisture and the amount of air entraining agent was increased from 2 ounces per yard to 2 1/2 ounces per yard. With these corrections, the mix became more workable and the slump changed from about 2 inches to about 3 1/2 inches. Both of our samples came from the first type of concrete. At 10:30 a.m. it was discovered that the median which had been poured between 8:00 and 8:30 a.m. was 1 inch low at the center of the span and tapered to the proper elevation at both ends as a result of an excessive deflection of the beam. Even though the concrete had set sufficiently to support a man's weight it was decided to raise the form and to top the concrete with additional concrete. This additional concrete was relatively stiff and the ultimate durability of this portion of the bridge is seriously questioned. The existing surface of the median which had been hand floated and edged was

wetted and concrete was placed on it at 11:20 a.m. and attempts were made to re-vibrate the two concretes together. During the general concreting operation the inspectors made a considerable number of slump tests and pressure air tests -- at least one from each load.

(4) Finishing. The finishing operation, which consisted of the longitudinal screed followed by a 5 inch canvas belt, was hampered in the initial stages by the relatively stiff concrete. The passes of the screed did not give a uniform closed surface and the absence of a float hampered the attainment of such a surface. The belt did contribute to achieving a reasonable surface. The efficiency of the finishing operation was considerably improved when the concrete mix was changed as noted above. When the concrete appeared too stiff to give a desirable finish some sprinkling was done with a whitewash brush. Brooming was not employed on this project and the final finish was applied with the canvas belting.

(5) Curing. Curing compound was applied with a hand spray soon after belting.

Summary. In general the finishing operation was severely handicapped by (1) an inadequate supply of concrete, and (2) a relatively stiff concrete.

As noted earlier, bridge #4 was overlaid to correct isolated deficient cover over reinforcement. It is of some significance to note that the higher corrosion potentials on the eastern end of the span, as shown in Figure 15, coincided with the end where the median was raised, etc.

Figure 32 is a view of the center portion of bridge #5 following the rainstorm, and Figure 33 a photograph taken in 1971 at approximately the same location. The primary defect is random cracking. The performance of the span under comparatively heavy traffic was somewhat better than would be expected. Sample 5-2 came from an area in the foreground of Figure 32. Figure 34 is a close-up of a vertical section through the surface. The intermingling of the curing compound with the reworked surface is obvious. Also obvious is the network of fine cracks which obviously will manifest themselves as pattern cracking of the surface. The termination of the cracks at the intersection with the curing compound below the surface is striking. This is consistent with the differences in absorption values in core 5-2 with depth as shown in Table 7, and the differences in chloride penetration for the two samples for bridge #5 shown in Figure 4. The isolated area of high corrosion potential shown in Figure 16 at the eastern corner of the slab was the area from which the screed rail was removed. Obviously, without knowledge of the special conditions during placement, variations in performance and in the properties of the concrete in the cores would be difficult to explain. It might also be observed that the rejection of the concrete and the lack of sufficient trucks due to the simultaneous placement of two decks resulted in delaying the job so that it was exposed to the thunderstorm. Had the delays not occurred, the deck would probably have been completed and cured with resulting satisfactory properties and performance.

Figure 32. View of span of bridge #5 following the rainstorm — 1963.

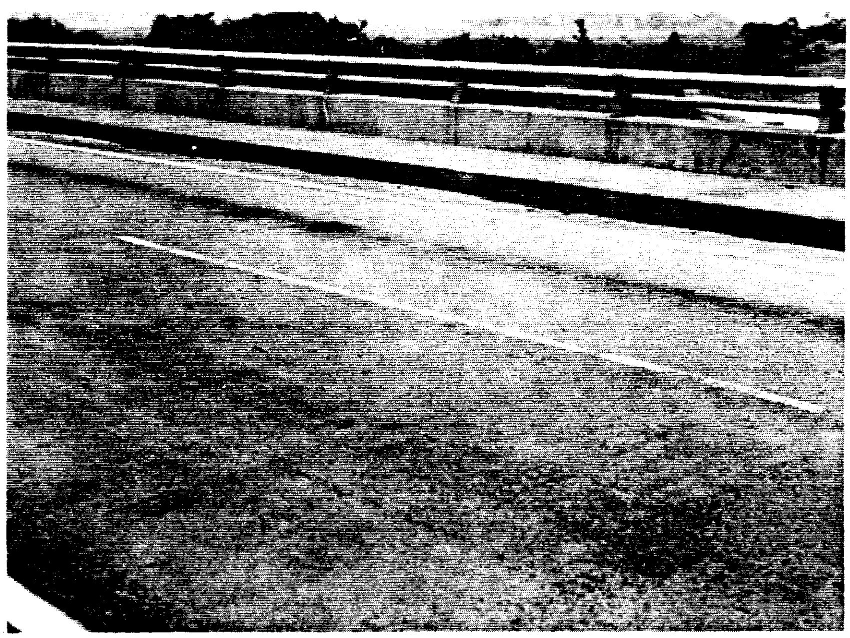


Figure 33. View of same area shown in Figure 32 — 1971.

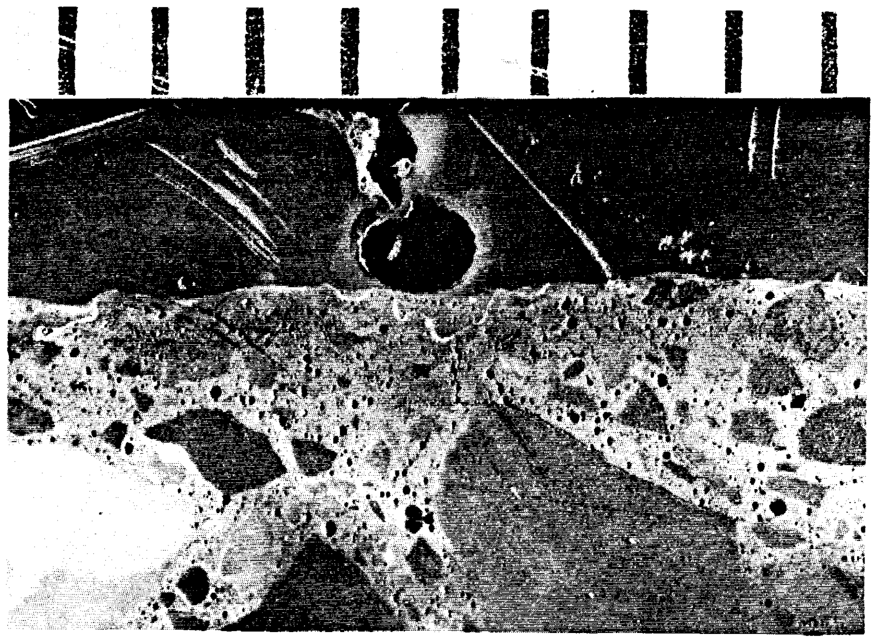


Figure 34. View of near surface of hardened concrete from sample 5-2. Note that fine cracking terminates at curing compound, which is intermixed with concrete. Large void at the top of the picture is in the epoxy mounting.

PERFORMANCE PREDICTED BY PETROGRAPHIC EXAMINATION AND SERVICE CONDITIONS

The air void characteristics determined from petrographic examinations have already been utilized in comparing the conventionally measured properties of concrete with the observed field performance. The objective of the petrographic examinations was to evaluate the microstructural features which correlated with the performance of the concrete and with other data such as absorption and chloride values.

The second objective of the entire project was to determine the levels of the data obtained from the samples of concrete that correlate with the condition of the concrete decks observed. It was hoped that definitions could be developed for the conditions of the concrete that would be predictive of the life of the decks and thus useful in making decisions on deck repairs and replacement.

In order that the correlations of air void data with absorption would be meaningful, the slabs used in obtaining the absorption data were employed in determining the air void parameters. The slabs were smoothed, finely lapped, dried, and subjected to analysis of the air void system according to the ASTM standard practice C457 using the linear traverse method. The linear traverse device employed a motorized lathe bed to move the lapped slab under the microscope and an electronic tallying system activated by push buttons. The magnification was 80x.

The results for each sample as the mean of all available levels within the sample are shown in Table 12, where they may be compared with the values obtained for fresh concrete with a pressure meter in the field.

Table 12

Air Contents as Measured in Fresh Concrete and Air Void Characteristics Determined by Linear Traverse of the Hardened Concrete

Sample	Percent of Air Fresh Concrete	Levels Available for Analysis[a]	Hardened Concrete Total Voids, %	Hardened Concrete Less Than 1 mm Voids, %	Hardened Concrete Spacing Factor $\bar{L}$, According to ASTM C457, in
1-1	3.7	(4,6,8)	6.58	2.92	0.00810
1-2	3.9	(2,4)	6.73	4.54	0.00740
12-1	3.6	(2,4,6)	6.12	4.12	0.00683
12-2	4.1	(2,4,6,8)	7.44	5.10	0.00600
2-1	7.9	(2,4,6)	11.63	8.59	0.00380
2-2	5.1	(2,4,6,8)	7.34	5.36	0.00580
5-1	2.0	(2,4,6,8)	5.64	3.48	0.00918
5-2	2.9	(2,4,6)	4.14	2.78	0.01023
15-1	4.0	(4,6)	2.91	1.81	0.01030
15-2	4.2	(4,6)	6.13	3.32	0.00830
16-1	5.6	(2,4,6)	5.74	4.19	0.00780
16-2	5.1	(2,4,6)	5.89	3.55	0.01000
3-1	2.7	(2,4,6)	5.42	3.44	0.01020
3-2	3.2	(2,4,6)	5.58	3.72	0.00970
4-1	2.0	(2,4,6)	3.78	1.93	0.01410
4-2	2.9	(2,4,6)	5.85	3.43	0.00940
9-1	3.4	(2,4)	4.42	2.61	0.01100
9-2	3.3	(2,4,6,8)	7.28	4.10	0.00890
14-1	6.0	(2,4,6)	7.03	4.04	0.00650
14-2	5.0	(2,4,6)	5.38	3.92	0.00750
17-1	4.0	(2,4,6)	7.19	5.50	0.00610
17-2	4.7	(2,4,6)	4.47	3.31	0.00880
6-1	4.4	(2,4,6)	7.28	4.62	0.00650
6-2	4.1	(2,4,6)	7.13	4.99	0.00700
7-1	4.0	(2,4,6)	8.61	4.78	0.00599
7-2	4.6	(2,4,6)	7.43	4.05	0.00770
8-1	2.9	(2,4,6)	6.01	2.54	0.01480
8-2	2.9	(2,4,6)	6.30	2.68	0.01257
10-1	3.6	(2,4,6,8)	7.87	3.75	0.00970
10-2	3.6	(2,4,6,8)	6.32	3.41	0.00930
11-1	5.7	(2,4,6,8)	7.05	5.17	0.00750
11-2	4.0	(2,4,6,8)	8.89	6.37	0.00490
13-1	4.6	(2,6)	4.65	2.93	0.01310
13-2	3.9	(2,4,6)	6.11	2.42	0.01540

[a] See Figure 1

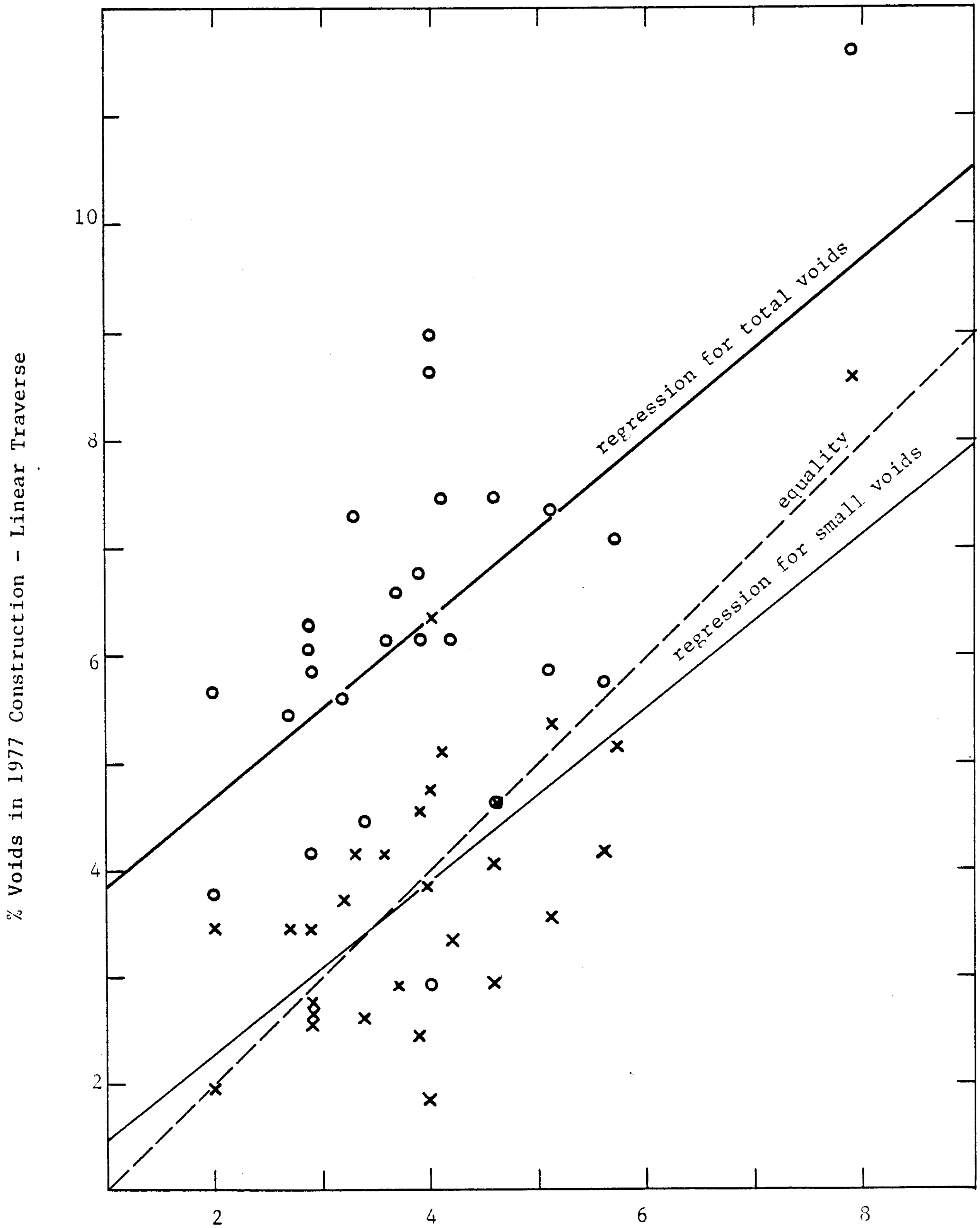
1 in = 25.4 mm

The total voids and entrained air voids are plotted in Figure 35 as functions of the air content of the freshly mixed concrete. The variability of the void contents was too great to warrant statistical evaluation, but approximate regression lines are shown. From these lines, it is seen that the air contents of the fresh concrete more closely approximate the entrained air voids than they do the total voids, the latter of which would be affected by consolidation and other construction procedures. This finding confirms similar results reported by Reidenouer and Howe (1974). While the direct agreement of air content and voids is rather poor, the ability of the air content to indicate the spacing factor, $\bar{L}$, in the hardened concrete is good, in that fresh concretes with air contents greater than 4.5% had acceptable spacing factors. This relationship is shown in Figure 36.

The performance data already discussed indicated that the deterioration exhibited by these bridges was influenced primarily by air voids in the 1½ in (38mm) of concrete below the wearing surface. The air void parameters determined for the top 1½ in (38mm) of concrete are listed in Table 13. The parameters included in Table 13 are those determined in accordance with ASTM C457. In addition a spacing factor $\bar{L}_s$ was calculated for small (entrained) voids according to procedures developed by Walker (1981).

The air void data were used to derive a factor of Air System Quality (AQ) which combined with a comparable factor describing the Paste Quality (PQ) provides a factor describing the Microstructural Quality (M_sQ). This last factor was used along with service conditions as a predictor of field performance.

The AQ was derived according to the point system given in Table 14.



O = total voids %
X = voids < 1mm observed diameter %

Figure 35. Void system of 1977 cores vs. air content of plastic concrete.

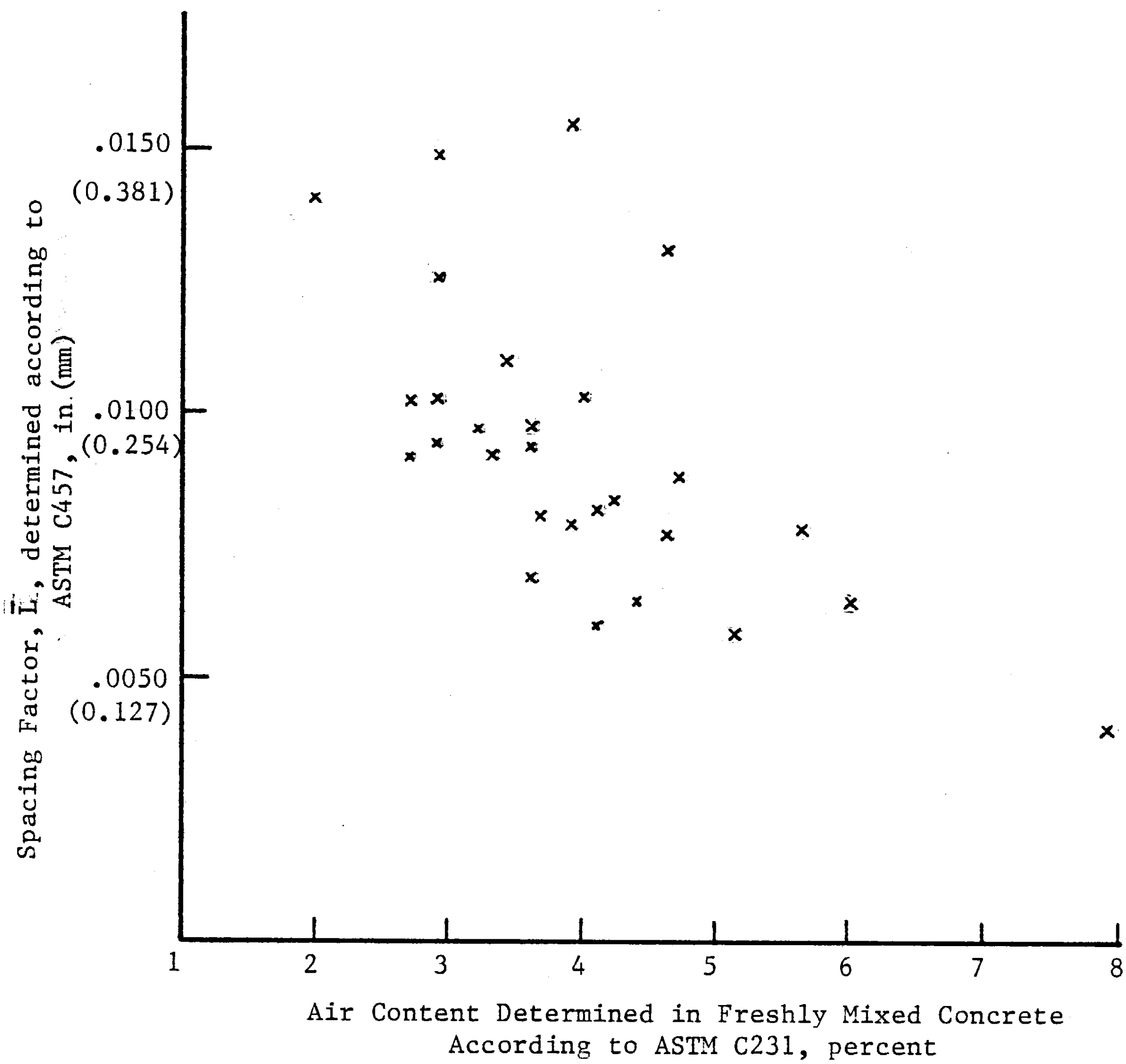


Figure 36. Relationship between Spacing Factor, $\bar{L}$, determined on hardened concrete and the air content measured by the pressure method at the time of placement.

Table 13

AIR VOLUME SYSTEM

Hardened Concrete -- Top 1½ in (38 mm)

Bridge Deck	Fresh Concrete Air Voids, Percent	Air Volume, %: Total Voids	Air Volume, %: Voids in Mortar	Air Volume, %: Voids < 1mm	Percent Voids > 1mm	Void Characteristics: Voids/in	Specific Surface in^2/in^3: Total	Specific Surface in^2/in^3: < 1mm dia.	Spacing Factor: ASTM C457 $\bar{L}$, in	Spacing Factor: Walker $\bar{L}_{sP}$, in
1-1	3.7	8.1	16.2	3.1	62	7.1	350	773	0.0078	0.0146
1-2	3.9	6.7	13.1	4.5	32	7.4	439	572	0.0074	0.0104
2-1	7.9	11.3	21.2	8.6	24	16.2	574	716	0.0039	0.0065
2.2	5.1	8.6	17.0	5.8	33	11.1	512	717	0.0052	0.0083
3-1	2.7	5.2	9.8	3.4	37	5.7	436	603	0.0106	0.0128
3-2	3.2	5.5	10.5	3.8	30	6.5	471	582	0.0099	0.0116
4-1	2.0	3.8	7.9	2.0	46	4.0	417	637	0.0128	0.0162
4-2	2.9	6.5	13.7	3.9	40	7.3	455	627	0.0087	0.0115
5-1	2.0	4.9	9.7	3.2	34	6.0	482	684	0.0102	0.0125
5-2	2.9	4.3	8.6	2.9	33	5.8	542	715	0.0099	0.0117
6-1	4.4	7.3	14.2	4.5	40	8.5	432	631	0.0071	0.0108
6-2	4.1	7.2	14.0	5.1	29	8.3	474	629	0.0077	0.0101
7-1	4.0	8.8	17.3	4.9	44	8.3	377	572	0.0056	0.0106
7-2	4.6	7.7	15.2	4.4	43	7.0	362	558	0.0079	0.0129
8-1	2.9	6.1	10.7	2.6	57	4.6	304	527	0.0145	0.0203
8-2	2.9	6.6	11.7	2.7	59	4.8	292	591	0.0130	0.0192
9-1	3.4	4.4	8.2	2.6	42	5.4	489	681	0.0110	0.0130
9-2	3.3	9.0	19.1	5.3	41	9.7	433	578	0.0088	0.0119
10-1	3.6	8.0	15.5	4.1	48	6.1	318	520	0.0097	0.0156
10-2	3.6	6.1	11.8	3.4	44	6.2	395	567	0.0094	0.0133
11-1	5.7	7.3	14.6	5.2	29	9.2	504	631	0.0078	0.0097
11-2	4.0	8.6	16.7	6.0	30	11.7	546	677	0.0051	0.0078
12-1	3.6	6.2	11.9	4.4	20	7.0	453	578	0.0063	0.0094
12-2	4.1	6.7	13.1	4.7	30	8.2	492	621	0.0060	0.0088
13-1	4.6	6.0	12.2	3.9	34	6.1	407	560	0.0096	0.0123
13-2	3.9	6.7	13.1	2.4	64	4.1	243	457	0.0160	0.0251
14-1	6.0	6.6	12.9	4.4	33	9.4	572	738	0.0063	0.0086
14-2	5.0	5.3	10.3	4.0	25	8.0	580	700	0.0078	0.0089
15-1	4.0	4.2	9.6	2.4	42	5.3	510	791	0.0102	0.0125
15-2	4.2	7.2	14.2	3.3	54	7.2	402	748	0.0066	0.0120
16-1	5.6	5.8	11.7	4.1	29	7.8	541	722	0.0084	0.0098
16-2	5.1	5.7	11.2	3.5	38	5.8	416	601	0.0102	0.0132
17-1	4.0	7.3	14.2	5.5	24	9.9	560	685	0.0058	0.0078
17-2	4.7	5.0	9.7	3.7	26	6.1	536	655	0.0086	0.0099

Note: 1 in = 25.4 mm; 1 in^2/in^3 = 0.04 mm^2/mm^3

Table 14

Air Void Parameters and Points Assigned in Calculating the Air System Quality Factor

Parameters	Value of Parameters	Points
Total Voids, %	$x < 3.5\%$	-10
	$3.5 \le x < 4.5\%$	0
	$4.5 \le x < 5\%$	20
	$5 \le x < 8\%$	25
	$8 \le x < 9\%$	20
	$9 \le x < 10\%$	10
	$x \ge 10\%$	0
Voids, %		$x < 2\%$ -10
	$2 \le x < 2.5\%$	0
	$2.5 \le x < 3\%$	20
	$3 \le x < 3.5\%$	25
	$3.5 \le x < 6.5\%$	30
	$6.5 \le x < 8\%$	20
	$8 \le x < 9\%$	10
	$x \ge 9\%$	0
Spacing Factor ASTM C457	$x \le .0075$ in	25
	$.0075 < \chi \le .0080$	20
	$.0080 < x \le .0100$	10
	$.0100 < x \le .01025$	0
	$.01025 < x \le .0110$	-5
	$.0110 < x \le .0130$	-10
	$.0130 < x$ -----	-15
Spacing Factor Proportional for Small Voids	$x < .0100$	25
	$.0100 \le x < .0110$	20
	$.0110 \le x < .0130$	0
	$.0130 \le x < .0150$	-10
	$.0150 \le x$	-15

In assigning points, the maximum value was given to void contents in the mid-range representing acceptable specification values. Lower volumes of air would detrimentally affect resistance to freezing and thawing and higher values would detrimentally affect strength and permeability and were thus assigned lower values.

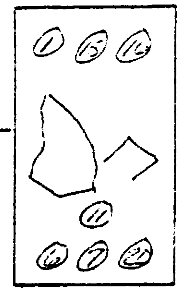
For an individual sample, the total points assigned could have a range between -30 and +105. Because few samples in which air entrainment was achieved consistent with good practice were represented, a range of from -25 to +120 was used to convert the actual values to a 0 to 100% scale in which 120 became 100% and -25 became 0.

To determine the PQ, two thin sections from each sample were impregnated with a flourescent dyed epoxy. The details of the fabrication methods may be found in an article by Walker and Marshall, (1979). Each of the two thin sections was cursorily examined. Twenty fields of view at a magnification of 400x were selected in the mortar areas. Each field of view was examined in detail, and data for each of 11 characteristics usually observed in petrographic examinations of mortar were recorded as shown in Figure 37 for section 4-2-A, an enlargement of which is shown in Figure 38. Each field of view was judged on each of the characteristics listed, and if the characteristic was important in that area of the mortar, the block was checked. The results for the two thin sections were counted and summed, and are presented in Table 15. Two of the characteristics, unhydrated cement and dense areas, were considered positive or beneficial factors because they are associated with low water-cement ratios. The remaining characteristics were assigned negative values because they are normally associated with high water-cement ratios.

When summed, the values assigned to the various characteristics gave values from +31 to -92. These values were converted to a percentage scale from 0-100% and this percentage was identified as the PQ. These values are shown in Table 15.

As previously noted the AQ and PQ were summed and reduced to a percentage designated the M_sQ as shown in Table 16. Also included in Table 16 are an environmental factor (EQ) and a traffic factor (TQ). These factors were derived from data given in Table 3. The EQ was calculated as a percentage of the most severe conditions of deicing and freeze-thaw cycling and summed as indicated in Table 16. The TQ is the average vehicles per day in thousands. While the AQ and PQ could be either positive or negative depending upon the quality of the concrete, the EQ and TQ would always be negative. The number listed in Table 16 as predicted durability is the sum of M_sQ, EQ, and TQ. The M_sQ values for the samples are plotted in Figure 39 and the sum of EQ and TQ for the decks in Figure 40.

Specimen Number 4-2-A #1



Sketch of Thin Section

# of Field	High Permeability Weak and/or Wet	Large Cracks	Small Cracks	$CaCO_3$	Large Cement Hydrated	Irregular Voids	Boundary Voids	Permeable Aggregate	$Ca(OH)_2$	Unhydrated Large Birefringent Cement	Dense Areas	Remarks
1	X		X	X	X	X			XX	X		
2			X	X	X				XX		X	abundant $Ca(OH)_2$
3	X		X	X					XX		X	
4	X		X	X	X		X		X			
5				X	X		X					
6		X	X	X	X	X						
7			X	X	X	X	X					
8				X	X		X				X	
9					X	X			X			
10					X	X			X			
11					X	X			X			
12					X				X			
13					X				X			
14				X								
15				X	X				X	X		
16			X		X		X					
17	X		X		X							
18					X					X		Very Large Void
19			X		X				X			
20			X		X		X		X			
TOTALS	4	1	10	10	18	6	6		15	3	3	

Figure 37. Worksheet used to determine PQ for specimen 4-2-A.

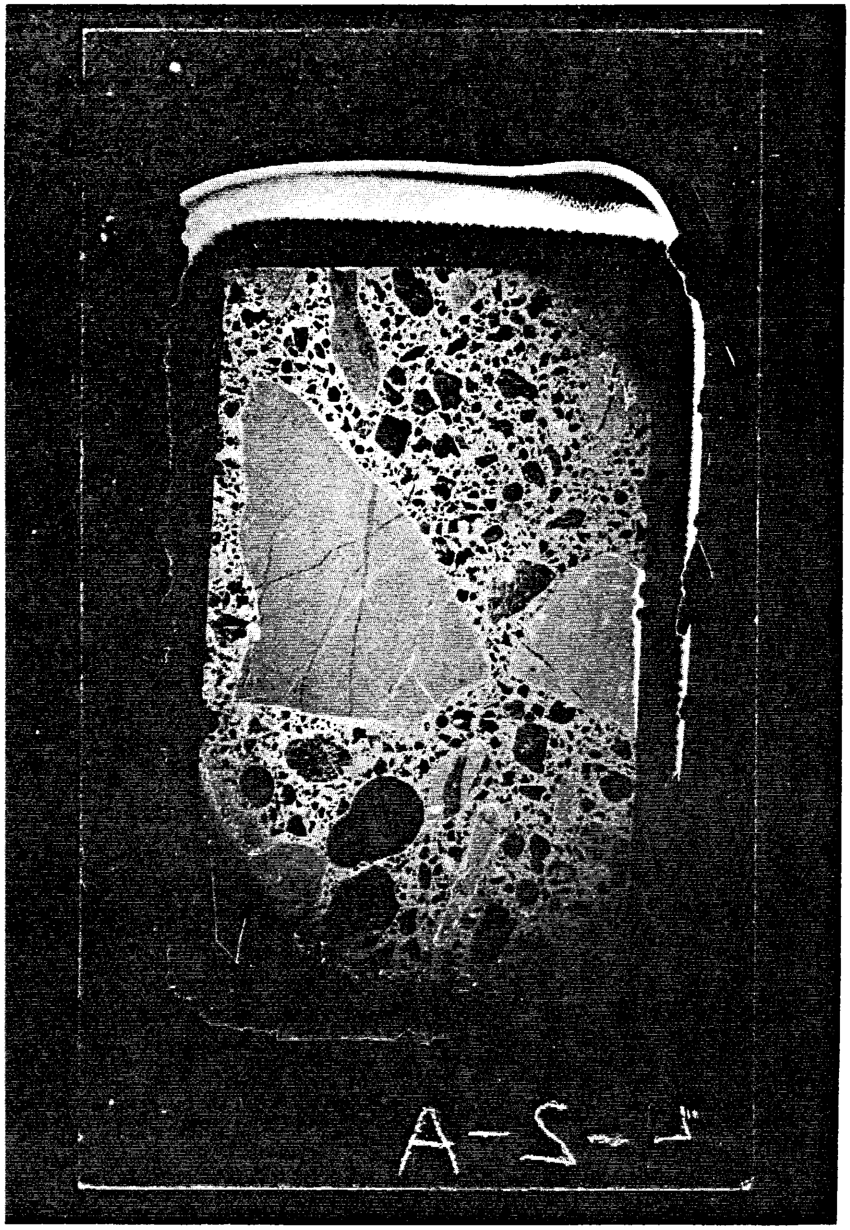


Figure 38. Thin section of concrete mounted on glass with epoxy. The section is 18 μm thick. The work sheet for this section was shown in Figure 37.

Table 15

Ratings of Properties of Paste from Thin Sections and Calculations of Paste Quality

Specimen	Cement Unhydrated, Percent[a]	+ #/2 Dense Areas	-# Internal Only Carbonation Factors	-# Factor for Large Cracks	-#/2 Factor for Small Cracks	-[(#-1)/2] Excess $Ca(OH)_2$ Factors	Factor for Weak Wet Areas[b]	Sum	PQ as % of Best
1-1	79	+4	-6	-7	-9	-18	-30	+13	85
1-2	65	0	-4	-13	-1	-11	-33	+3	77
2-1	29	0	-40	-2	-5	0	-16	-34	47
2-2	59	+5	-25	-1	-3	0	-4	+31	100
3-1	52	+3	-9	-1	-5	-16	-22	+2	76
3-2	24	+1	-22	-1	-7	-15	-52	-72	16
4-1	50	+8	-5	-9	-8	-9	-25	+2	76
4-2	17	+13	-12	-3	-9	-13	-35	-52	33
5-1	30	-4	-4	0	-5	-4	-27	-6	70
5-2	36	+1	-25	-6	-7	0	-16	-17	61
6-1	13	+1	-32	0	-3	-1	-53	-75	14
6-2	55	+3	-3	0	-3	-10	-31	+11	84
7-1	5	+3	-22	0	-6	-6	-17	-43	40
7-2	5	+2	-13	-6	-7	-10	-18	-47	37
8-1	0	0	-10	-1	-5	-14	-26	-56	29
8-2	15	0	-4	-1	-1	-20	-25	-36	46
9-1	3	0	-2	-2	-1	-17	-25	-44	39
9-2	10	0	-40	-3	-6	-6	-47	-92	0
10-1	15	+1	-21	-7	-10	-8	-16	-46	37
10-2	49	+1	-27	-5	-6	-15	-5	-8	68
11-1	2	0	-23	0	-10	-3	-28	-62	24
11-2	3	0	-6	0	-3	-6	-62	-74	15
12-1	8	+2	-18	-2	-10	-11	-27	-58	28
12-2	17	+3	=9	-4	-6	-6	-64	-69	19
13-1	20	+1	-20	-3	-9	-11	-30	-52	33
13-2	25	+2	-5	0	-2	-8	-37	-25	54
14-1	3	+1	-3	-7	-6	-6	-30	-48	36
14-2	6	+5	-5	-8	-5	-7	-15	-29	51
15-1	5	0	-8	-3	-9	-15	-21	-51	33
15-2	9	0	-6	-8	-5	-15	-20	-45	38
16-1	60	+7	-14	-1	-8	-5	-18	+21	92
16-2	43	+3	-4	-2	-5	-1	-10	+24	94
17-1	21	+2	-38	-3	-7	-2	-30	-57	28
17-2	0	0	-5	-6	-2	-1	-66	-80	10

= number of checks indicating importance from both thin sections

a [Unhydrated/(Unhydrated + Hydrated)] x 100 = Cement Unhydrated Percent

b -[2(Weak Wet) + # Boundary Voids] = Weak Wet Area Factor

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Table 16

Calculations of Predicted Durability

Specimen	Paste Quality	Air Quality	Microstructural Quality	Environmental Factor	Traffic Factor	Predicted Durability
1-1	85	55	80	-80	-18	-18
1-2	77	86	92	-80	-18	-6
3-1	76	52	74	-42	-1	+30
3-2	16	62	47	-42	-1	+4
5-1	70	45	66	-66	-10	-10
5-2	61	38	58	-66	-10	-18
6-1	14	86	59	-64	-19	-24
6-2	84	83	94	-64	-19	+11
7-1	40	83	71	-64	-20	-13
7-2	37	69	62	-64	-20	-22
8-1	29	28	36	-38	< 1	-3
8-2	46	31	46	-38	< 1	+8
10-1	37	52	53	-58	< 1	-5
10-2	68	52	69	-58	< 1	+11
12-1	28	90	68	-86 [a]	Never Opened	-17
12-2	19	90	63	-86 [a]	"	-22
13-1	33	62	56	-98	-5	-47
13-2	54	14	42	-98	-5	-61
15-1	33	17	32	-63	-12	-43
15-2	38	69	62	-63	-12	-13
16-1	92	79	96	-67	-27	+2
16-2	94	52	83	-67	-27	-12
17-1	28	90	68	-55	< 1	+12
17-2	10	79	51	-55	< 1	-5
			Have been overlaid			
2-1	47	62	63	-100	-10	-47
2-2	100	86	100	-100	-10	-10
4-1	76	0	46	-67	-4	-25
4-2	33	72	61	-67	-4	-10
9-1	39	17	35	-80	-6	-50
9-2	0	52	33	-80	-6	-53
11-1	24	90	66	-63	< 1	+2
11-2	15	90	61	-63	< 1	-2
14-1	36	90	72	-58	-7	+7
14-2	51	90	80	-58	-7	+15

[a] Deicer calculated as 2/3 that of adjacent lane.

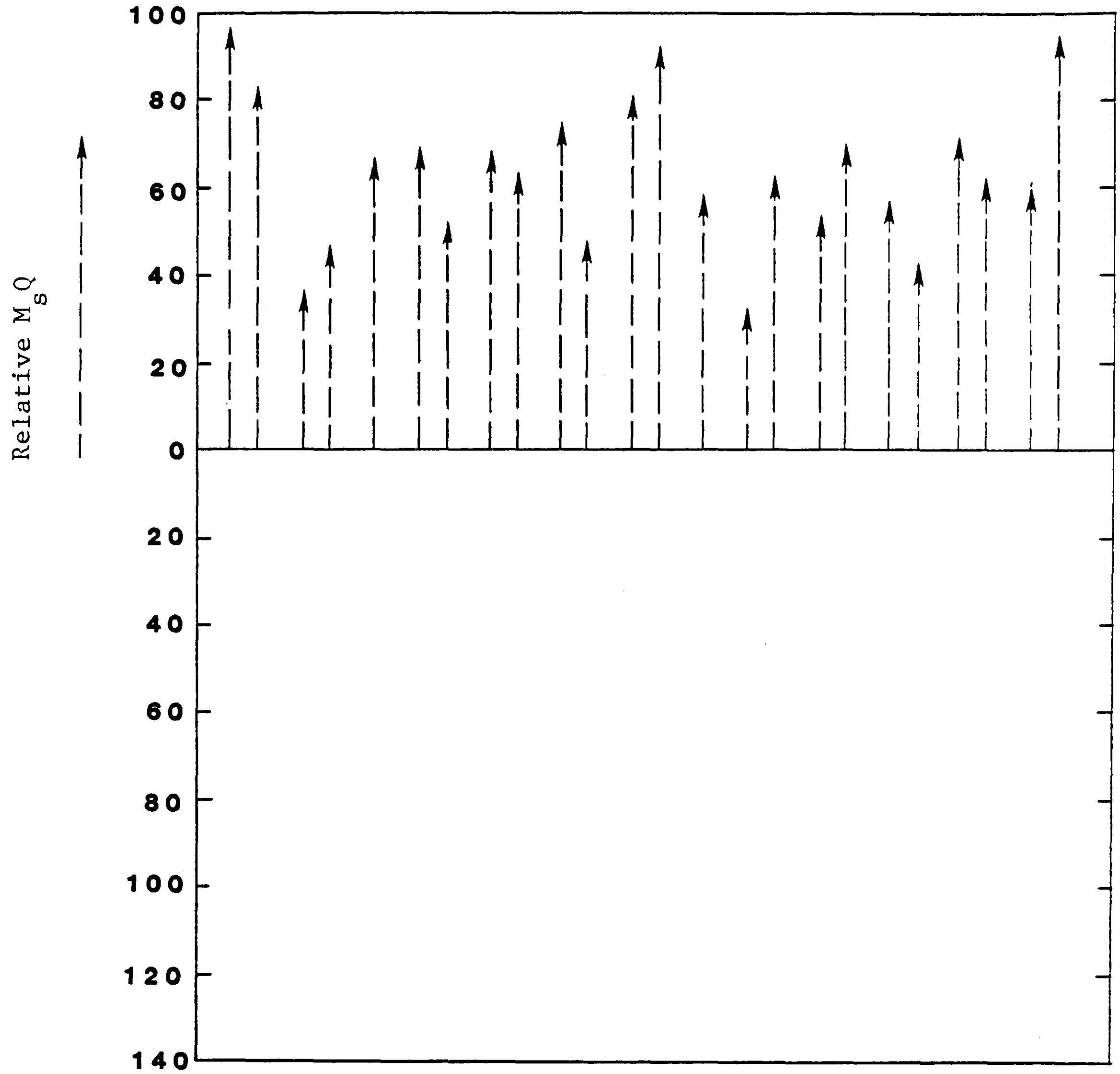

Figure 39. Non-overlaid bridges arranged in order of increasing deterioration. Microstructured quality plotted in vertical direction from 0 line.

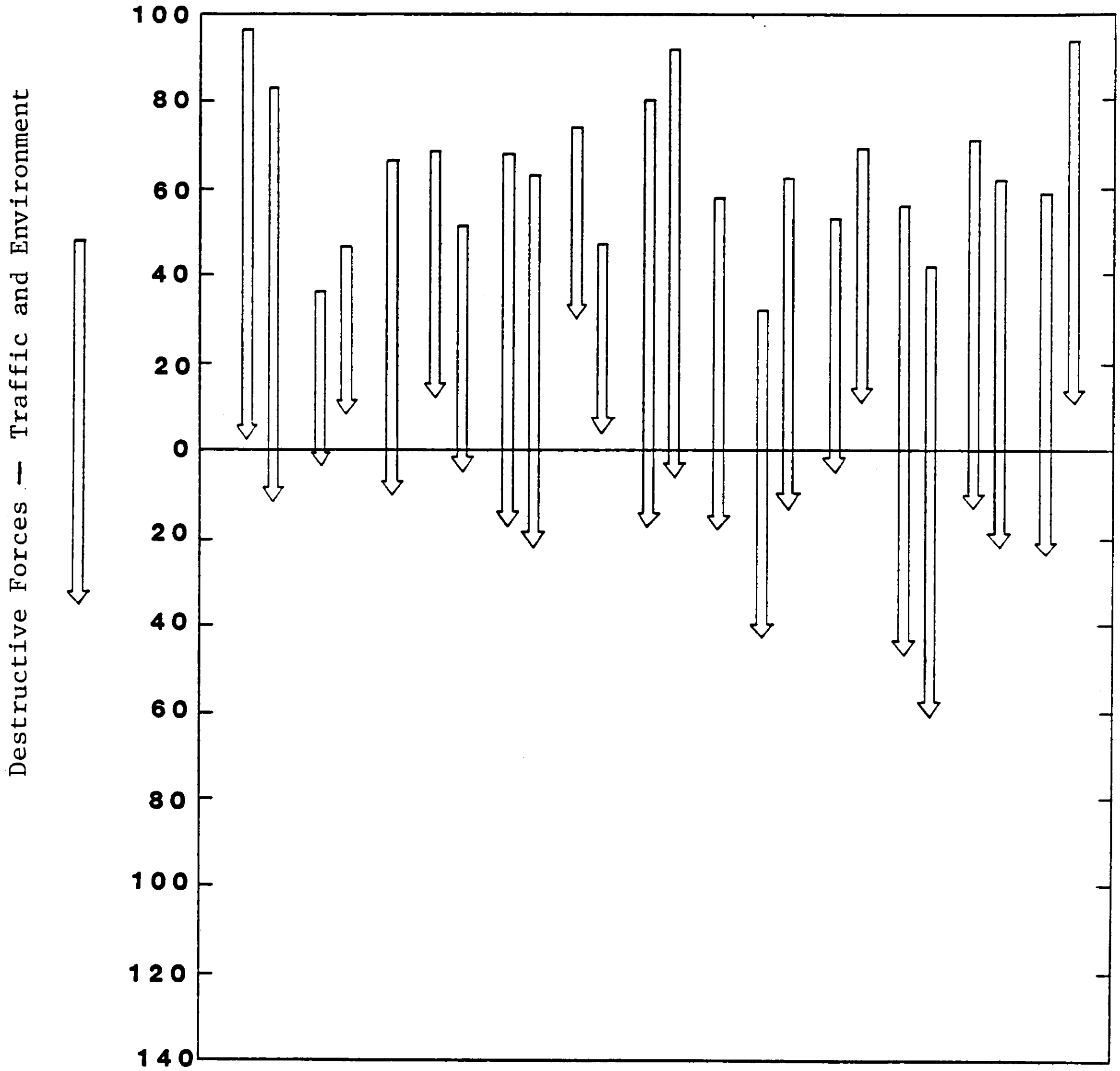

Figure 40. Non-overlaid bridges arranged in order of increasing deterioration. Sum of EO and TQ plotted downward from end points in Figure 39.

The condition of the decks determined from visual observation was shown earlier in Tables 4 and 5. For comparison with the predicted durability, the data in these tables for the uncovered spans were converted to a numerical scale of from 0 to -100, with the best bridge (#16) being 0 and the worst (#6) being -100. Use of this scale gave the performance curve shown in Figure 41. In this figure, the bridges are plotted in order of increasing deterioration from left to right. The measures of AQ, PQ, EQ, TQ, and performance are combined in Figure 42. For each bridge specimen, beginning at zero the M_sQ values shown in Figure 39 were plotted in a positive direction. From the end point of the M_sQ, the deleterious factors EQ and TQ are plotted in a negative direction to the point of the arrow at the predicted durability value. The stepped band surrounding the field performance plots is two standard deviations or errors of estimate in width as calculated from the correlation of the predicted durabilities with the field conditions. Bridges #6 and #7 were excluded from this calculation, because, as mentioned earlier, being silicone treated bridges they belong to a different population. The predicted durability arrow points for bridges 6 and 7 are much higher than the field conditions. No negative factor for silicone treatment was included in this diagram. Such a factor would have to be assigned from 30 to 70 negative quality points to make the prediction sufficiently close to the values for field conditions.

Similarly, no negative factor was included for the uncertain cure condition and unusual nature of bridge #10. The graph indicates that a negative factor of 20 would be sufficient.

The data shown earlier in Table 3 for bridge #3 did not reflect the fact that it carries mainly trucks. The data available are not sufficient for calculation of how they should be weighted. The large difference in M_sQ values for the two specimens from bridge #3 makes any estimate of the proper factor for truck traffic very uncertain, but it appears that a factor of -10 would be realistic. This factor would also give consideration to the relatively large amounts of deicing compounds that truck tires carry from the nearby truck stop.

It appears that Figure 42 shows rather clearly what would be anticipated from the properties of the concretes and service conditions; namely, that in most cases durability will be a direct function of the balance between the quality of the concrete and the forces of deterioration.

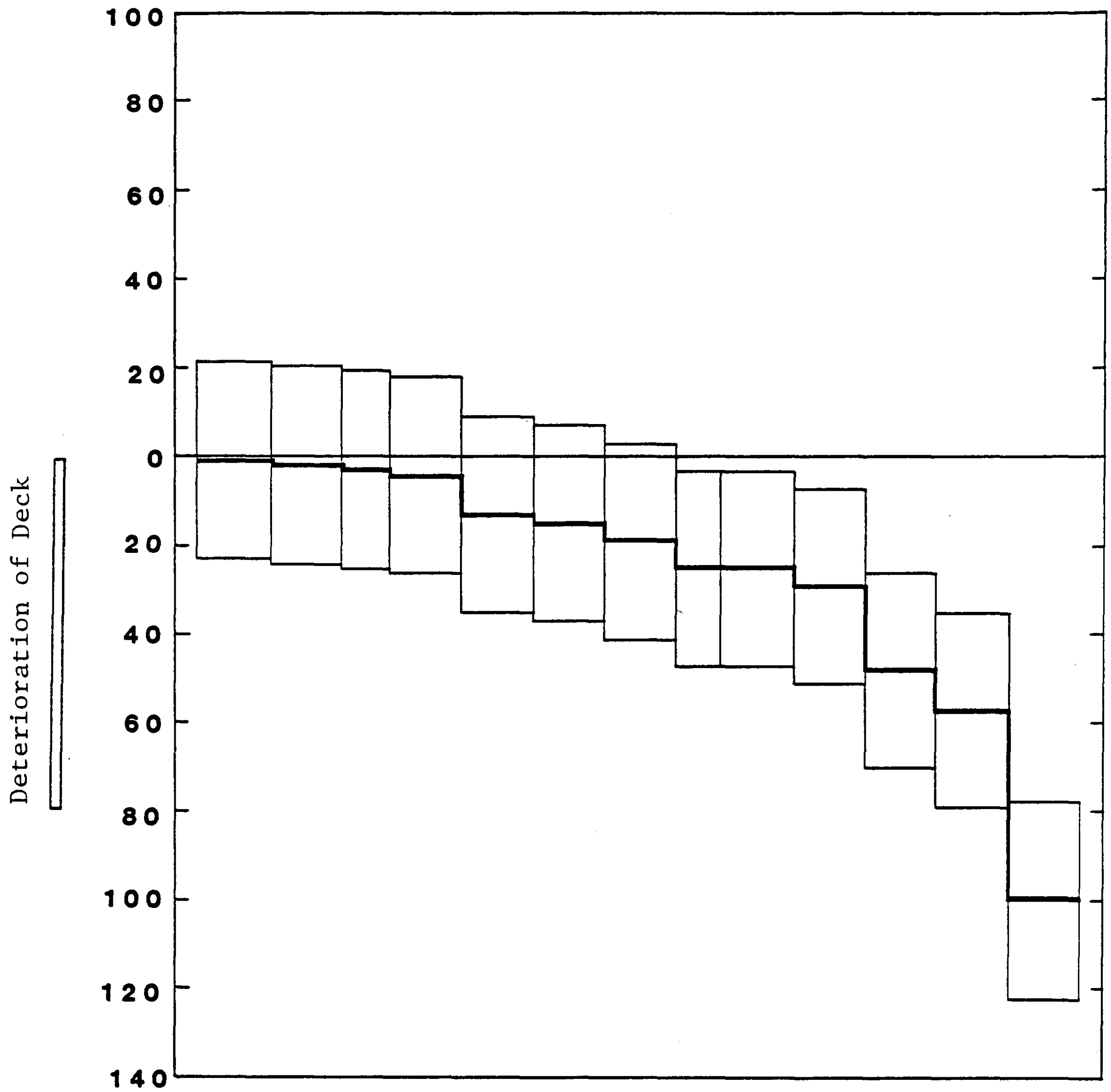

Figure 41. Non-overlaid bridges arranged in order of increasing deterioration. Relative amount of scaling and cracking shown by heavy black line plotted on scale of 0 to -100.

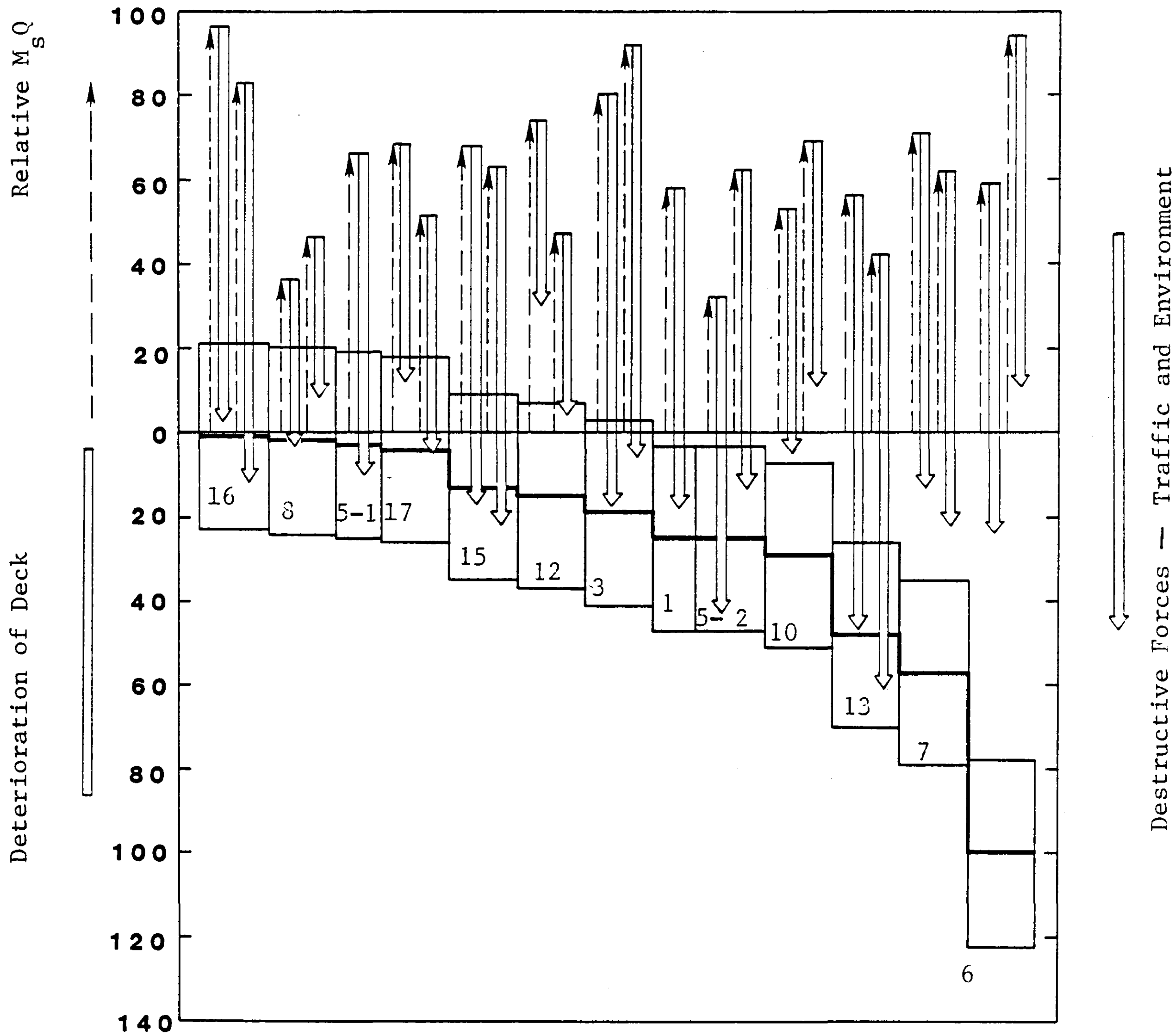

Figure 42. The balance between the quality of the concrete and the forces of deterioration.

The relationship of various parameters of the concrete to the quality factors used in this investigation and to the parameters of the air void system was investigated mathematically by means of linear regression. The coefficients of correlations were in most cases rather low and the "t" test was used to indicate the confidence with which one can accept the various relationships. The results of these calculations are presented in Table 17.

Notice the positive signs in the correlation at the 1/2-in (13-mm) level for total voids and voids over a millimeter in diameter. The levels of confidence seem to indicate that the permeability at this level is deleteriously influenced by an abundance of larger voids. At a 1½-in (38-mm) depth the presence of large voids appears to become of less importance and the total volume of voids appears beneficial. The correlations of chloride related deterioration and absorption with the other air void parameters seem to support this conclusion.

One might question how low the absorption must be to prevent chloride intrusion at the depth of the steel. Table 17 shows a confidence level of 95% for the relationship between the chloride ratio at 1 3/4 in (44 mm) and the absorption at 2 to 4 in (50 to 100 mm). This linear regression curve shows that the standard error of the estimate for absorption is 0.5%. Zero chloride intrusion correlated with 4.5% absorption. Thus, 4.5 ± 1.0% (or 3.5% to 5.5%) is a desirable absorption level. It may, therefore, be concluded that absorptions of less than 3.5% have an excellent chance of having prevented chloride intrusion to the 2-in (50-mm) level in 10 year old concrete.

It is recognized that the relationships shown in Figure 42 are based upon limited data and might not be applicable to a larger sample. The results are encouraging, however, and suggest that the approach be applied to a larger sample, perhaps including a measure of permeability in the M_sQ and refining some of the other parameters. If the approach were to prove valid for a large sample it would be a valuable tool, not only for assessing action with regard to nonspecification concrete in place but also in making judgments as to the remaining service life of a deck as part of a bridge management system.

Table 17

Confidence With Which The Relationships Listed Can Be Accepted

	Ratio of Chloride at 3/4" to Surface Chloride	Absorption ¼-½"	Ratio of Chloride at 1 3/4" to Surface Chloride	Absorption at 2-4" Depth
Absorption ¼-½"	95% (+)	---	---	---
Chloride ratio 1 3/4"	99.9% (+)	80% (+)	---	---
Absorption 2-4"	98% (+)	88% (+)	95% (+)	---
$M_s Q$	95% (-)	98% (-)	95% (-)	80% (-)
AQ	80% (-)	80% (-)	80% (-)	60% (-)
PQ	80% (-)	95% (-)	90% (-)	40% (-)

	1/2" Level Only		Mean to 1½" Depth	Not Investigated
Total voids > Vol.%	20% (+)	60% (-)	60% (-)	
Voids Vol. % > 1mm seen dia.	80% (+)	80% (+)	20% (+)	
Voids Vol. % < 1mm seen dia.	60% (-)	20% (-)	60% (-)	
No. voids/in	80% (-)	60% (-)	95% (-)	
No. voids < 1mm/in	90% (-)	60% (-)	95% (-)	
% by vol. of voids > 1mm	90% (+)	60% (+)	40% (+)	
Specific surface	90% (-)	95% (-)	40% (-)	
Specific surface, small only	60% (-)	98% (-)	80% (-)	
Spacing factor C457	60% (+)	60% (+)	90% (+)	
Spacing factor proportional, small only	80% (+)	80% (+)	60% (+)	

Note: 1 in = 25.4 mm

SIGNIFICANCE OF RESULTS

Even though there are comparatively few decks in the study sample, considering the number of variables that influence the field performance of concrete the various measures of performance are practically without exception relatable to the characteristics of the concrete observed during construction or in cores subsequently removed for testing. The prediction of performance as indicated in Figure 42 is exceptionally good.

The tremendous attention that has been focussed on "the bridge deck problem" in recent years has tended to create the illusion that the increased deterioration observed nationwide might be caused by factors beyond solution through traditional approaches. The results from this study suggest that adequately consolidated and properly cured concrete with low water-cement ratios and good air void systems have shown resistance to scaling and chloride penetration for 14 years under heavy traffic and exposure to deicing chemicals. In this light it is of interest to note that the earliest systematic studies of concrete in America during the twentieth century were conducted by Duff A. Abrams of the Lewis Institute in Chicago, which later became the Portland Cement Association. Bulletin No. 1 contained Abrams' famous water-cement ratio relationship (Abrams 1918); Bulletin No. 2 was titled "Effect of Curing Conditions on Wear and Strength of Concrete" (Abrams 1919[a]); and Bulletin No. 3 discussed the "Effect of Vibration, Jigging and Pressure on Fresh Concrete" (Abrams 1919[b]). While Abrams was concerned primarily with strength, his first efforts were directed at three of the most important elements in the production of concrete: the water-cement ratio, curing, and consolidation. The specifications under which the decks studied in this present research were constructed would be considered inadequate by today's standards. However, those decks that met all the requirements in force at the time they were constructed have performed adequately. From these data, it would appear that decks that will give adequate performance can be built by complying with the guidelines in reports of the American Concrete Institute's Committees on Durability (ACI 201 1977), Consolidation (ACI 309 1971) and Curing (ACI 308 1977). For decks, ACI Committee 201 on Durability recommends a maximum water-cement ratio of 0.45 and air contents of 6% ± 1½%. Current requirements of the Virginia Department of Highways and Transportation conform to these recommendations.

It is true that the rates and volumes of modern construction impose formidable problems for quality assurance systems that must ensure that the requirements for the four vital elements evident from this study are consistently met. Statistical approaches are necessary, but they depend upon performance requirements or "end-result tests." While numerous instrumental methods for determining water-cement ratios have been extensively studied the results have generally been disappointing, in terms of either accuracy or complexity. While a method such as the Kelly-Vail procedure for determining water-cement ratios is sufficiently accurate for purposes of highway construction, it is comparatively expensive and thus difficult for an agency like the Virginia Department of Highways and Transportation to adopt, since on a given day concrete might be placed at perhaps fifty locations distributed over an area of almost 41,000 sq mi (106 x $10^9 M^2$). It might be possible, however, to have the method used on particularly important placements. Tests for adequacy of curing have been developed (Carrier and Cady 1970) but are not widely used and are thus of uncertain utility. Nuclear methods for measuring the adequacy of consolidation have been widely studied, but experience in Virginia indicates that they can detect only gross examples of poor consolidation rather than the differences that seem important based upon petrographic studies of hardened concrete (Ozyildrim 1981).

Fortunately, adequate testing methods for air entrainment are available, but considerable reliance for ensuring a proper water-cement ratio, consolidation, and curing must be placed upon the expertise and number of job inspectors. Recent trends toward reducing the number of inspections, encumbering inspection personnel with numerous administrative details, etc., have resulted from economic restraints, but the results from this study suggest that considerable maintenance expenditures will be required in the future if inspection is reduced beyond the levels necessary to assure strict compliance with the requirements for placing quality concrete. Of the 17 decks in this study, 6 have required resurfacing in less than 15 years, and at least 5 more will probably require resurfacing before about 20 years of service. The 3 decks that most closely met all the required conditions give no evidence that they will require maintenance within the near future.

CONCLUSIONS

Based upon the results from this study the following conclusions are warranted.

A. Relationship of Performance to Initial Properties of Concrete (applicable to new construction)

   1. Resistance to scaling and chloride penetration has been maintained for 14 years by concrete that met the requirements established by ACI Committee 201 on Durability. The few decks that met these criteria give no evidence that this resistance will be reduced in the foreseeable future.

Concretes that did not meet these requirements have shown reduced resistance to scaling, chloride penetration, and other defects. Six of the 17 decks have required overlays and others will in the near future for reasons clearly related to concrete quality and construction techniques.

Upgradings of materials and construction specifications made since 1963 have been consistent with ACI recommendations, with the exception that the specified maximum water-cement ratio was 0.47 until April 1983. Since then the value of 0.45 suggested by ACI has been used.

2. The importance of long-established principles for producing durable concrete are confirmed by the results of this study, in that the overriding variables of importance to chloride penetration are cover depths, water-cement ratio, consolidation, and curing. These field results confirm the controlled experiments of Clear (1976).

   For resistance to scaling, the water cement-ratio plus proper air entrainment were clearly demonstrated to be of most importance.

3. Three of the four structures constructed with the longitudinal screed have been overlaid because of deficient cover over the reinforcing steel. Revised procedures initiated as a result of research by Hilton (1971) since construction of these decks are directed toward overcoming this problem.

4. The use of a silicone treatment has resulted in increased scaling on both bridges to which it was applied. This indication of the detrimental influence of the treatment is strengthened by the fact that a nearby deck that has concrete with the same characteristics except for the treatment has performed well. The higher moisture content attributable to the sealing effect of the treatment is confirmed by the corrosion potential measurements on these two bridges.

5. The use of an epoxy bond in the two-course construction on bridge #14 appears to have reduced the chloride penetration on this structure.

6. The prediction of performance from data obtained from petrographic examinations of the concrete combined with service conditions shows promise for use in evaluating the performance of concrete in decks.

B. General Relationships

A significant relationship was found between the resistance to chloride penetration and the absorption of the concrete. In concrete that had an absorption of below about 4.5% and had been in service 14 years of field service no chloride has reached the level of 1 3/4 in (44 mm). When the absorption is greater than this value, the chloride levels increase directly with absorption. This finding implies (a) that absorption is related to the combined influences of the water-cement ratio, consolidation, and curing: (b) that under service conditions in Virginia, the major entry of chlorides into bridge decks is with bulk water rather than by ion migration which is important for continuously wet structures; and (c) laboratory evaluations in which concrete is kept in continuous contact with a chloride solution may not reflect field performance in Virginia as well as would tests involving intermittent periods of drying.

C. Relationship of Performance to Properties (applicable to maintenance)

1. The conditions for corrosion continue to exist in several decks that have been overlaid.

2. Data from corrosion potential measurements are consistent with data on materials, construction, and traffic and were reproducible and consistent on the four bridges tested in 1971 and again in 1977.

3. Data from corrosion potential measurements and chloride contents must be used together to assess the condition of a deck. Either used alone could result in a misleading interpretation.

4. Corrosion activity as indicated by half cell measurements was documented for the case of no chloride or low chloride levels in concrete where one end of the reinforcement extended into heavily contaminated concrete. This has important implications, not only for overlaying decks but also for patching in which new chloride free concrete is placed in areas adjacent to contaminated concrete.

D. Evaluating the Potential Durability of Concrete that Fails to Meet Specification Requirements

1. In addition to the usual characteristics evaluated, including core strengths and air void parameters, an absorption test appears to be the best measure of the combined effects of the

water-cement ratio, consolidation, and curing. When the absorption of concrete is below 4.5% the potential resistance to chloride penetration appears to be good, while above this value it appears to be poor. The conclusion stated in A-6 applies to maintenance considerations as well, and might be particularly valuable as part of a bridge management system.

RECOMMENDATIONS

A. Department Procedures

1. The current specification requirements for bridge deck concrete (Class A-4) are adequate and consistent with applicable guidelines of the American Concrete Institute, and the comparatively recent reduction of the maximum allowable water-cement ratio to 0.45 was warranted.

2. Continued emphasis should be placed upon improving the number, expertise, and available time of inspectors on bridge deck placements. If economic and administrative restrictions do not permit this action, then consideration must be given to designating especially critical deck placements for intensive inspection, including requiring the district concrete technician to be present during placement. The criteria for judging criticality might be based upon anticipated traffic volumes that could increase the likelihood that early maintenance would be required if inspections were to be insufficient.

3. The use of half cell potential measurements in concert with chloride content data is the best currently available technique for assessing the potential for corrosion of reinforcing steel and the two types of data must be used together for deck evaluations.

4. In cases of dispute as to the potential durability of concrete whose conformance to specification requirements is in doubt, the best available bases for a decision are a proper air void system ($\overline{L} < 0.0080$) and an absorption value less than 4.5%, pending confirmation of the approach using petrographic data as developed in this study.

B. Research Needed

1. The relationship between absorption and the water-cement ratio, consolidation, and curing should be refined.

2. The validity of predicting performance from data developed through petrographic examinations and anticipated service conditions should be tested for a larger sample of concrete properties and service conditions than was used in this study.

ACKNOWLEDGEMENTS

Because this project has extended over a considerable period of time and drawn heavily upon the results from previous Council projects, many people have contributed in various ways, including the gathering of field data, preparing and analyzing laboratory samples, and the typing and editing of the various reports. Most of the major contributors are included as authors in reports listed in the References. Appreciation is expressed to them as well as to the technical and support staff involved. One person, Clyde F. Giannini, Materials Technician Supervisor, who has been involved in a significant way in all of these studies, including the initial installations and coordinating and conducting the field surveys, deserves special recognition for his contribution.

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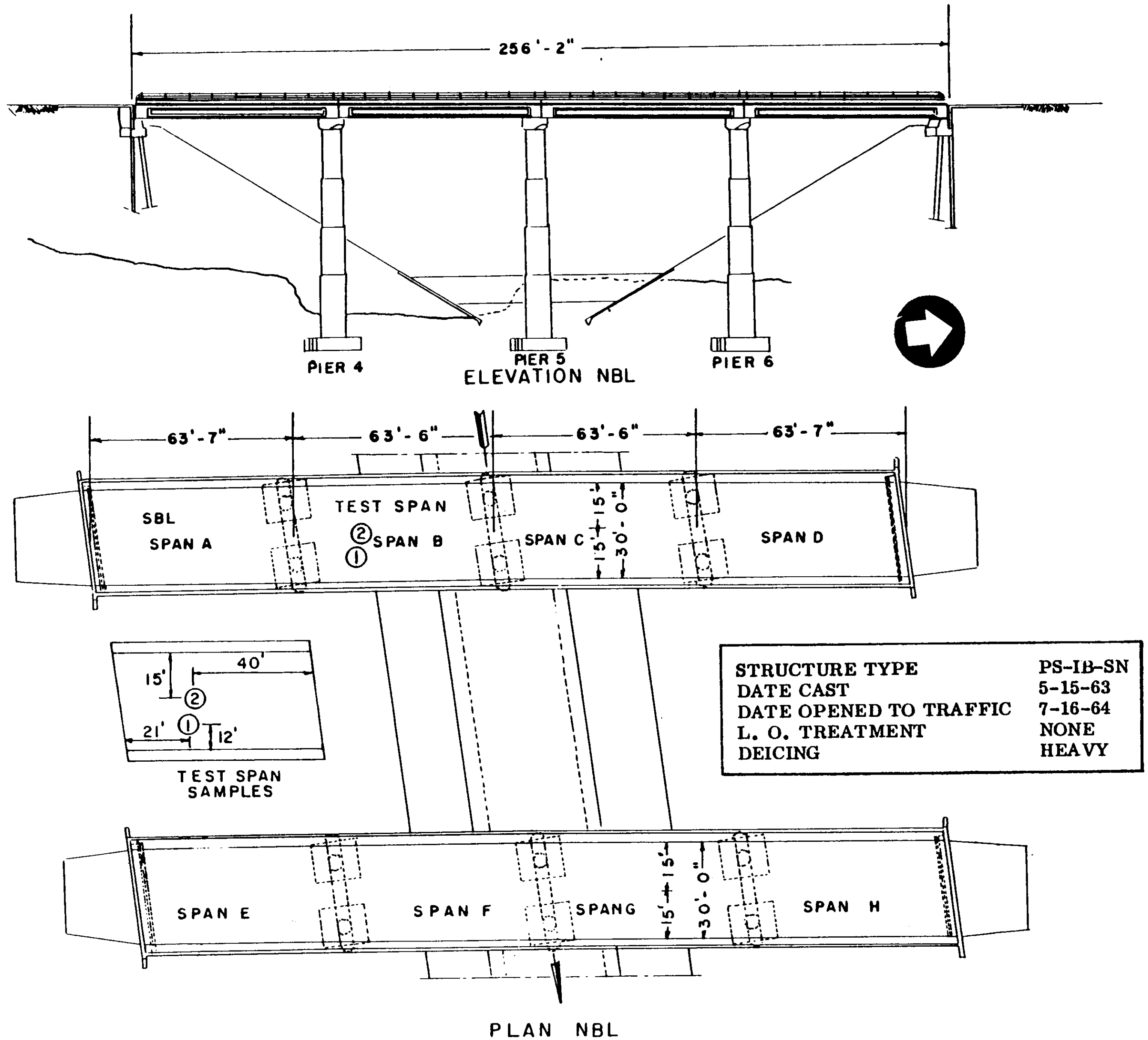
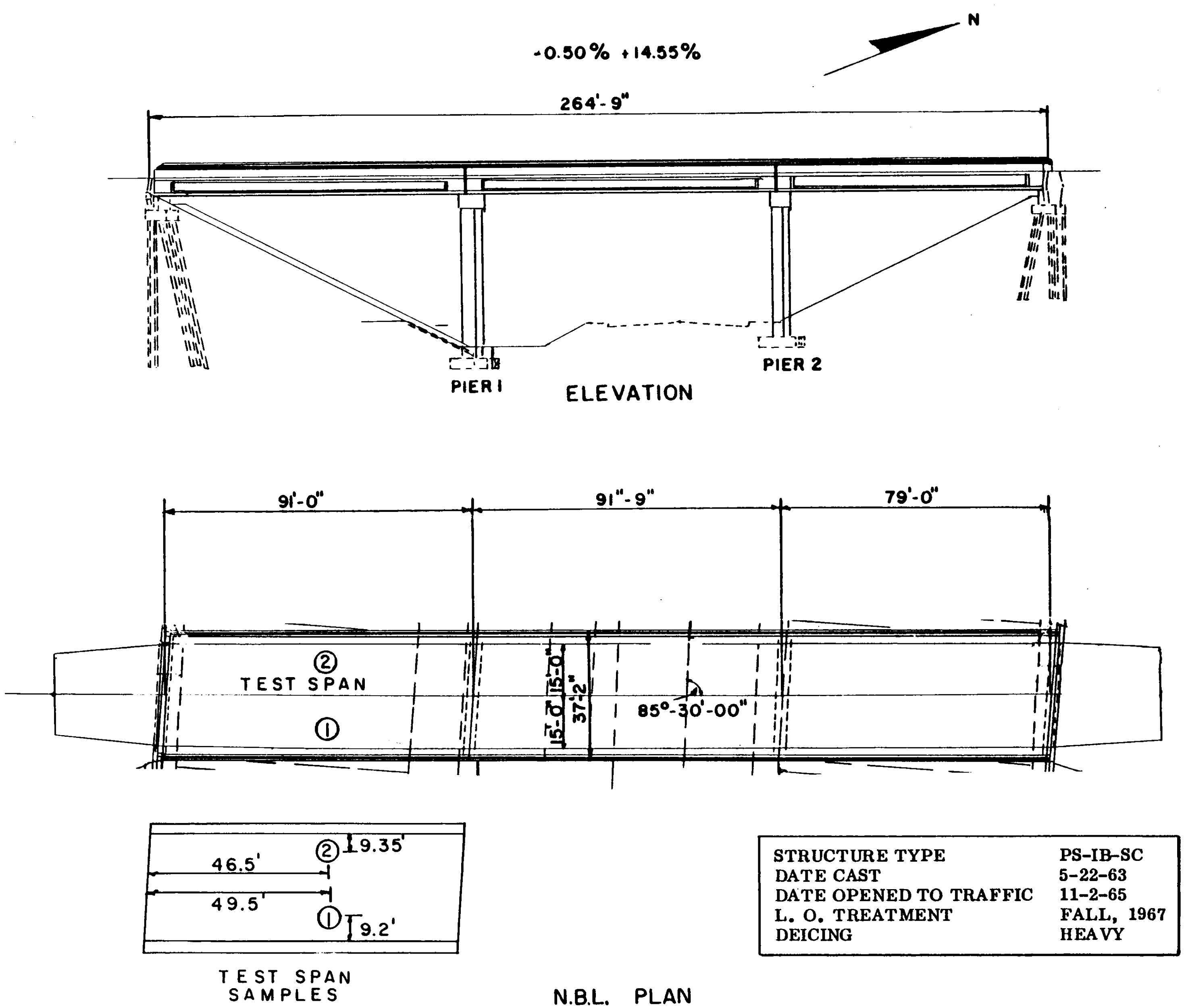
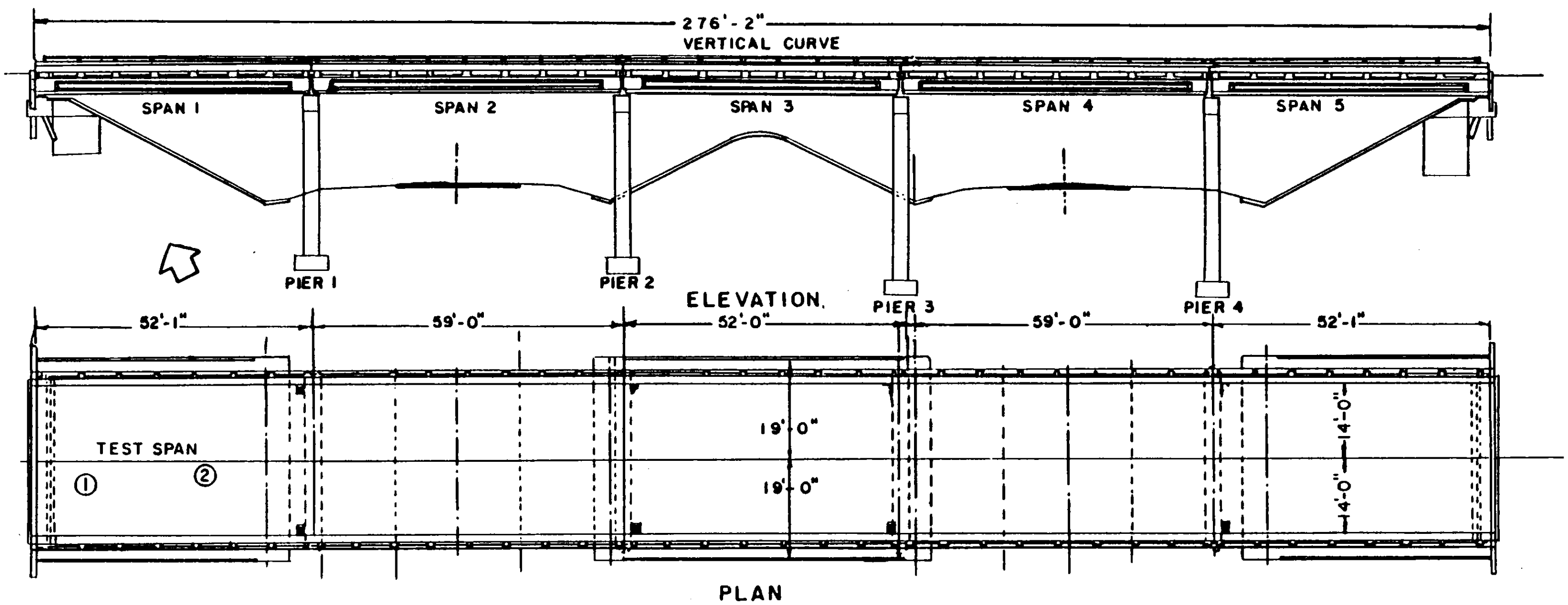


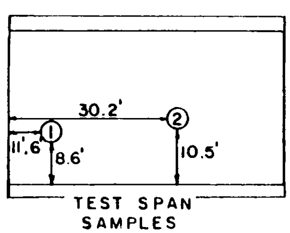
Figure A-1. **Plan for bridge #1; also shown are the locations of test samples.**



STRUCTURE TYPE	PS-IB-SC
DATE CAST	5-22-63
DATE OPENED TO TRAFFIC	11-2-65
L. O. TREATMENT	FALL, 1967
DEICING	HEAVY

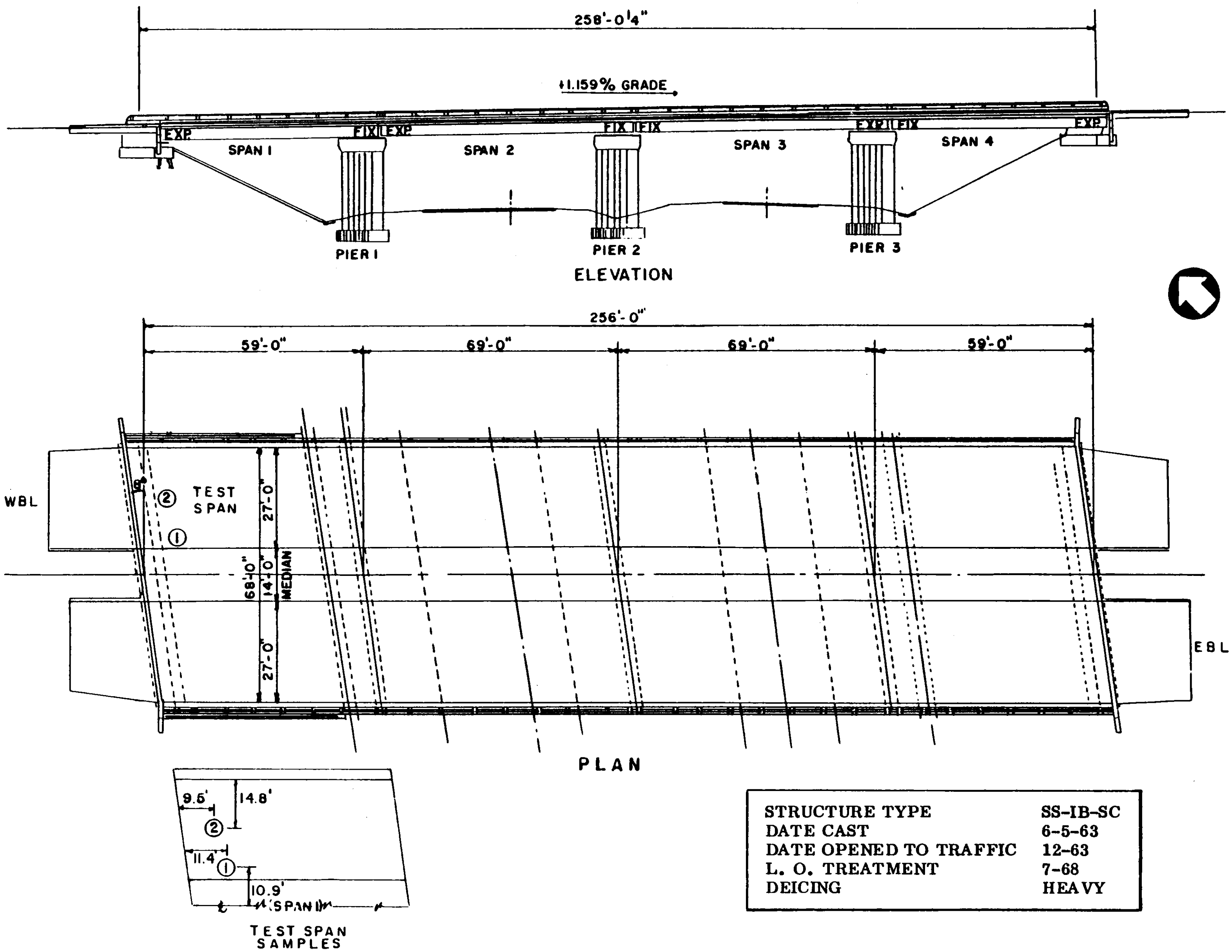
Figure A-2. Plan for bridge #2; also shown are the locations of test samples.





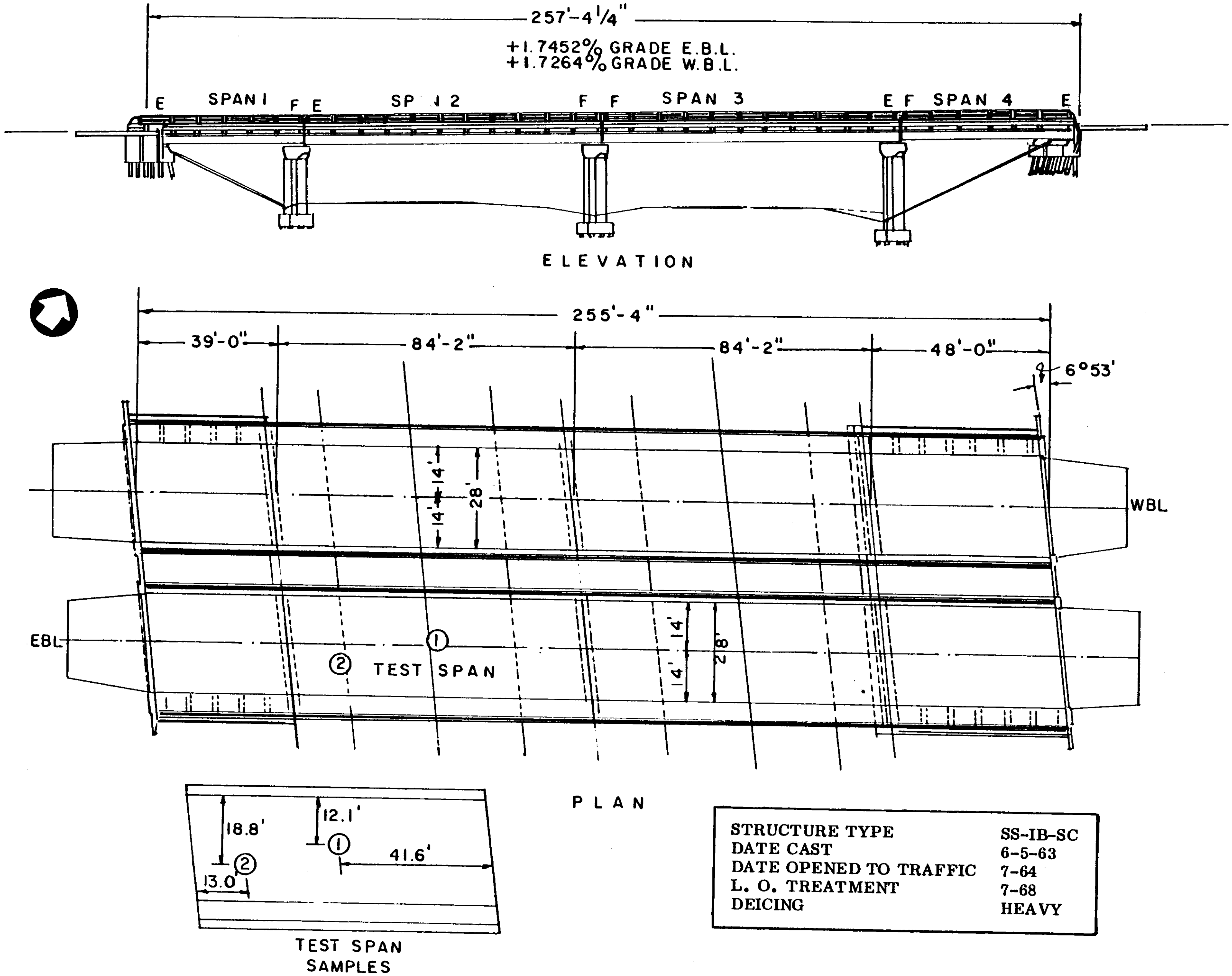
STRUCTURE TYPE	PS-IB-SN
DATE CAST	5-23-63
DATE OPENED TO TRAFFIC	12-16-65
L. O. TREATMENT	NONE
DEICING	MEDIUM

Figure A-3. **Plan for bridge #3; also shown are the locations of test samples.**



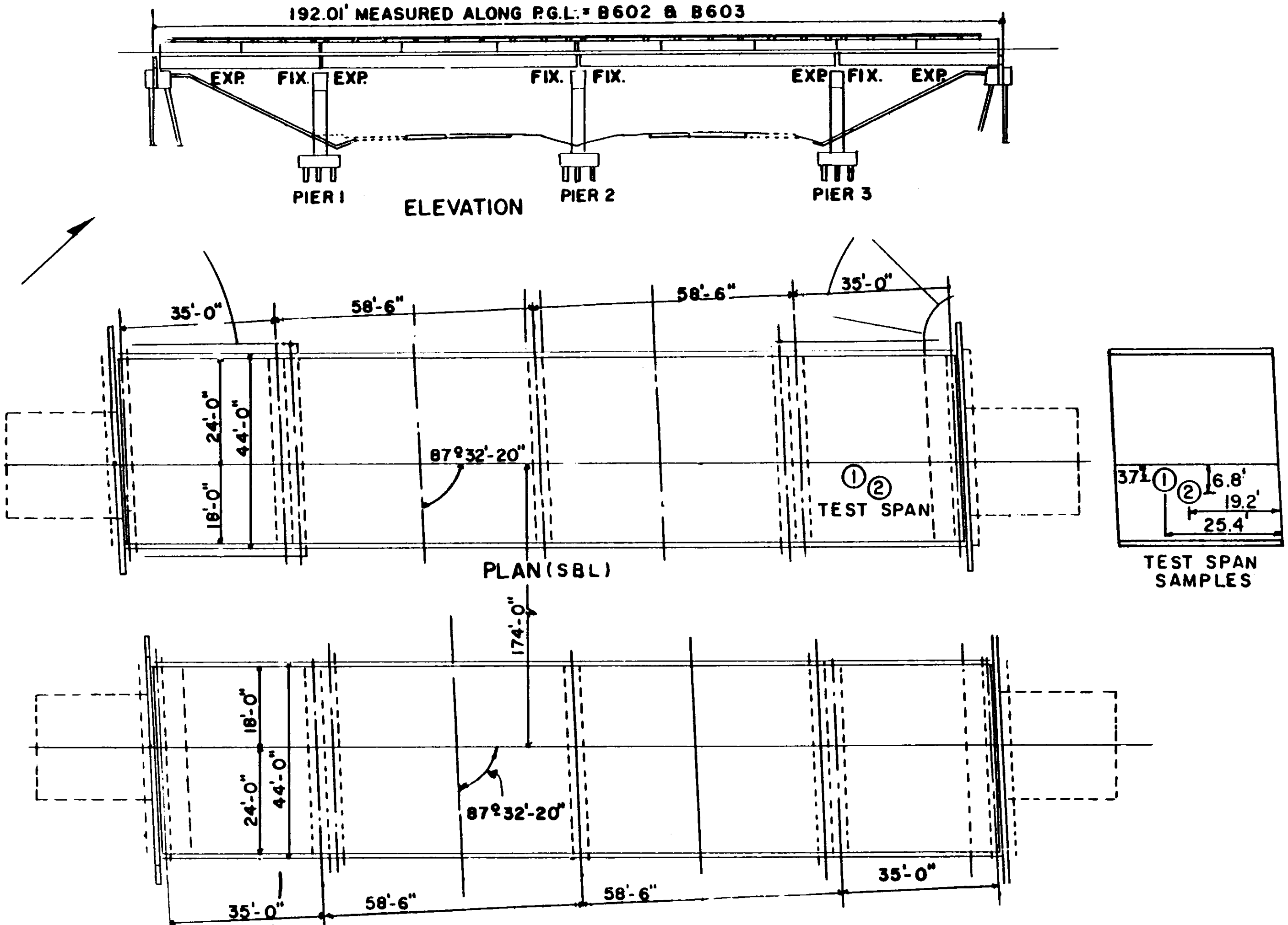
STRUCTURE TYPE	SS-IB-SC
DATE CAST	6-5-63
DATE OPENED TO TRAFFIC	12-63
L. O. TREATMENT	7-68
DEICING	HEAVY

Figure A-4. Plan for bridge #4; also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB-SC
DATE CAST	6-5-63
DATE OPENED TO TRAFFIC	7-64
L. O. TREATMENT	7-68
DEICING	HEAVY

Figure A-5. Plan for bridge #5; also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB-SC
DATE CAST	6-7-63
DATE OPENED TO TRAFFIC	12-18-64
L. O. TREATMENT	NONE*
DEICING	HEAVY

*INITIAL TREATMENT OF SILICONES

Figure A-6. Plan for bridge #6; also shown are the locations of test samples.

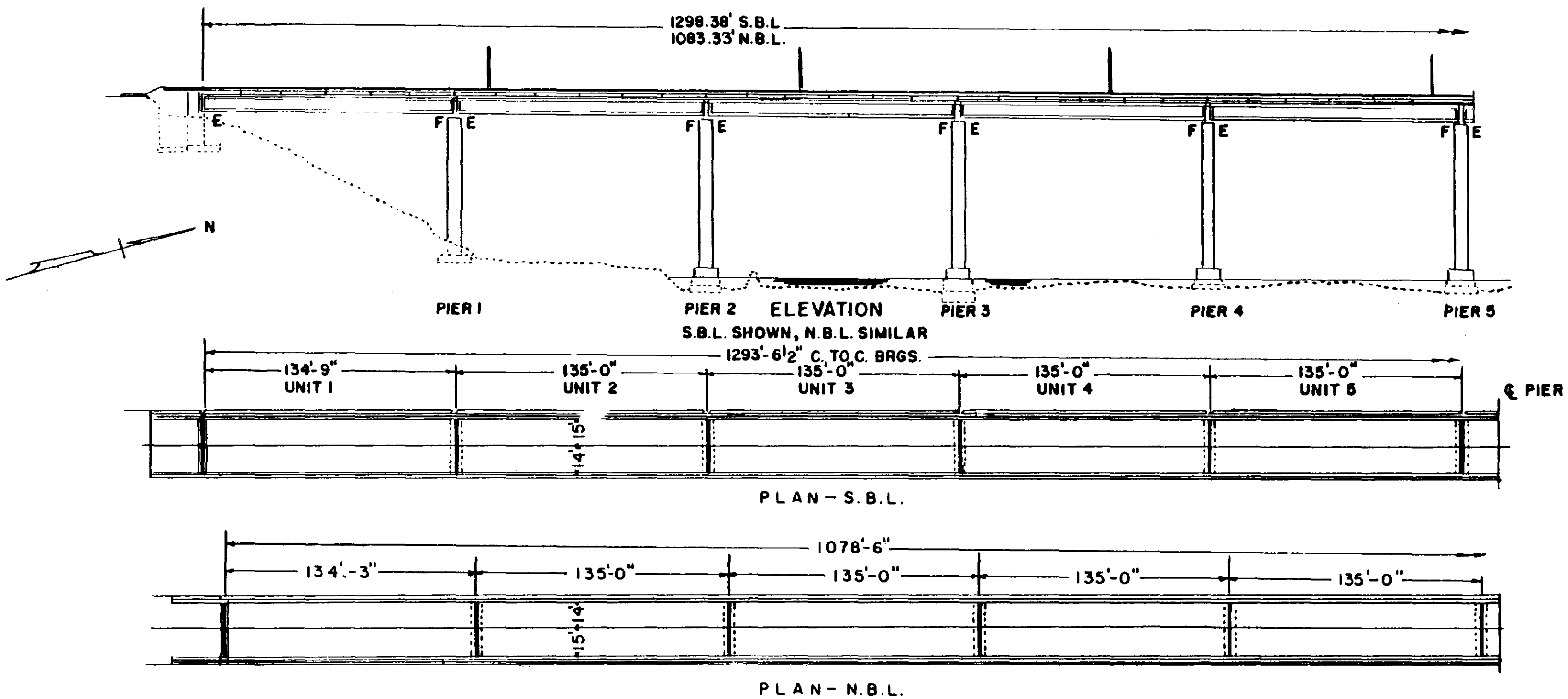
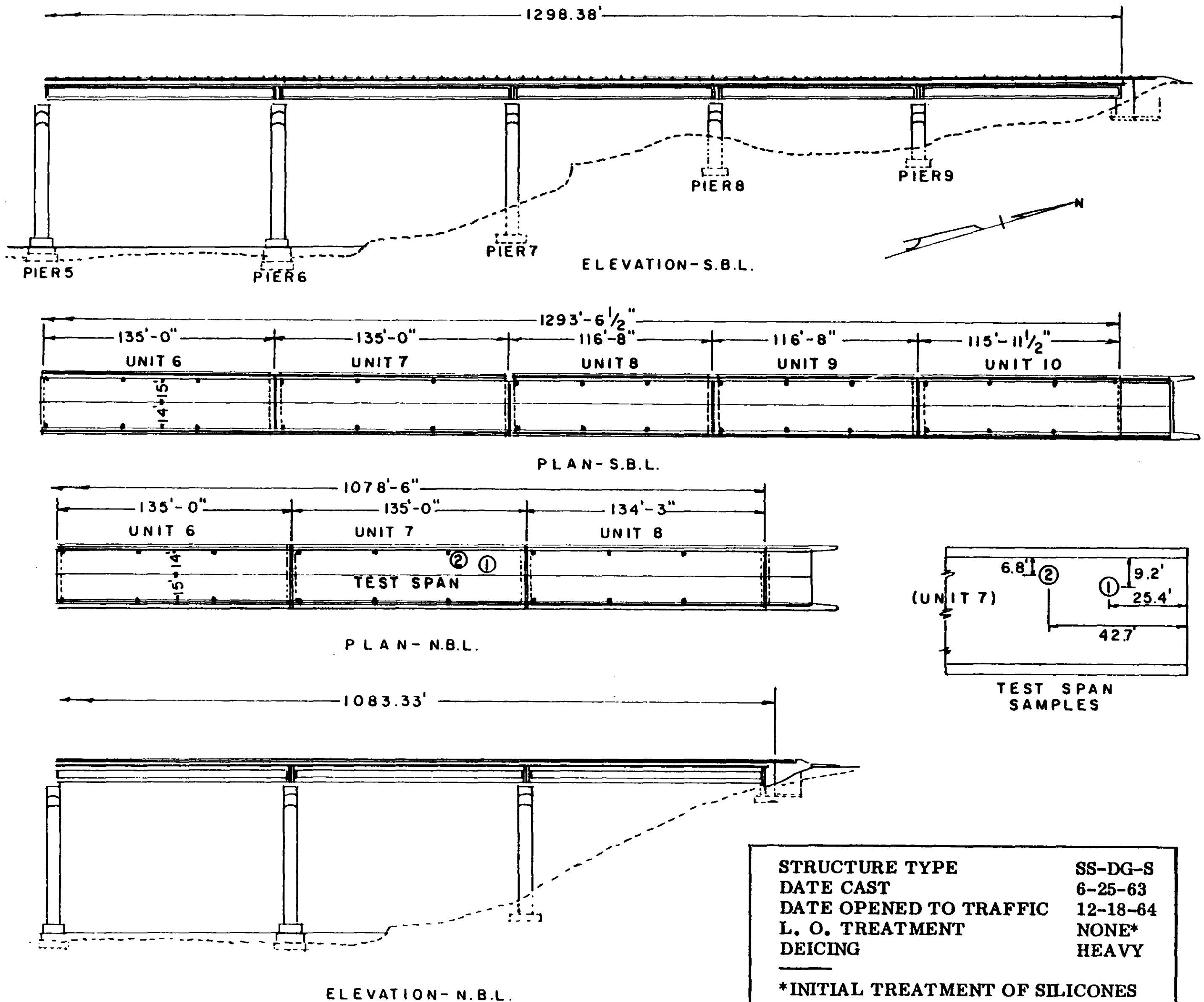


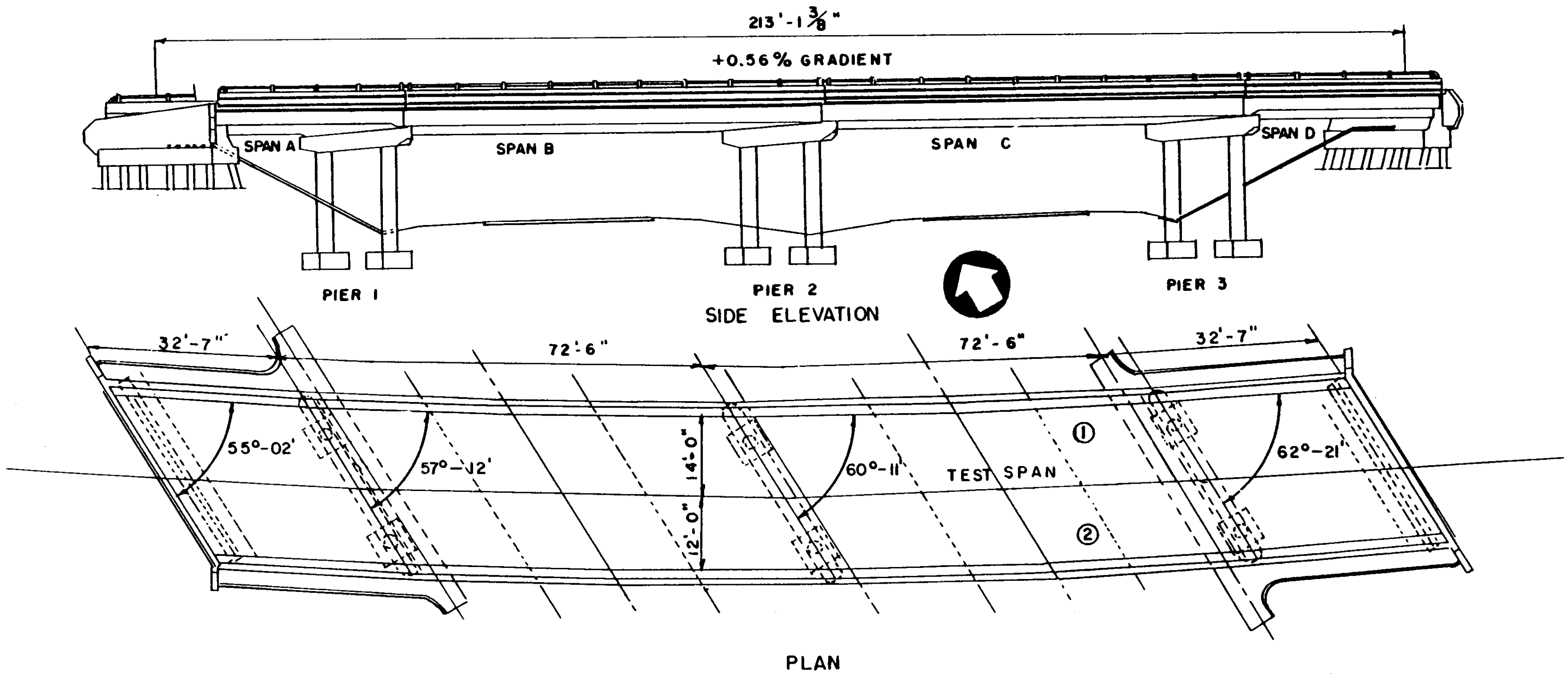
Figure A-7(a). Plan for bridge #7 (southern section); also shown are the locations of test samples.



STRUCTURE TYPE	SS-DG-S
DATE CAST	6-25-63
DATE OPENED TO TRAFFIC	12-18-64
L. O. TREATMENT	NONE*
DEICING	HEAVY

*INITIAL TREATMENT OF SILICONES

Figure A-7(b) Plan for bridge #7 (northern section); also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB-SC
DATE CAST	6-27-63
DATE OPENED TO TRAFFIC	11-18-63
L. O. TREATMENT	NONE
DEICING	MEDIUM

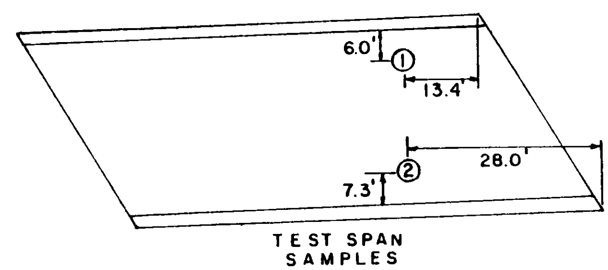


Figure A-8. Plan for bridge #8; also shown are the locations of test samples.

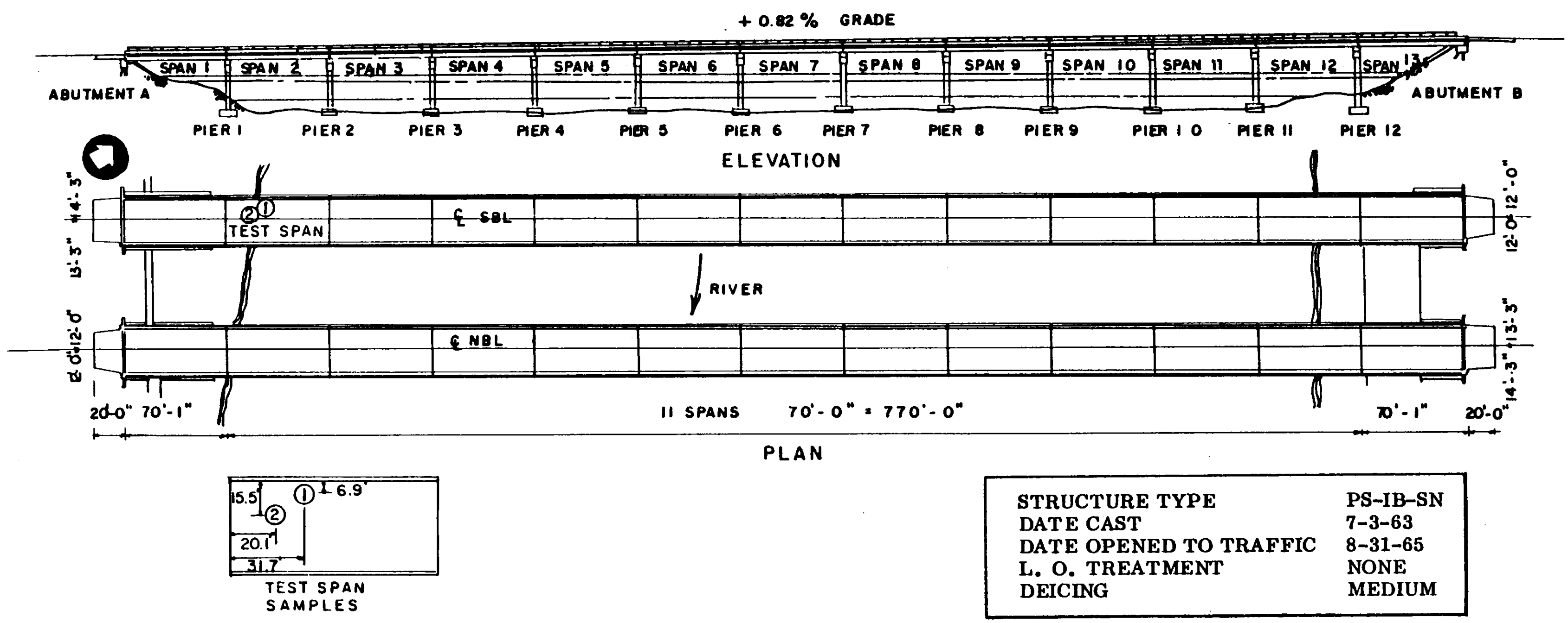


Figure A-9. **Plan for bridge #9; also shown are the locations of test samples.**

STRUCTURE TYPE	RC-SS-C
DATE CAST	7-11-63
DATE OPENED TO TRAFFIC	SPRING, 1964
L. O. TREATMENT	NONE
DEICING	LIGHT

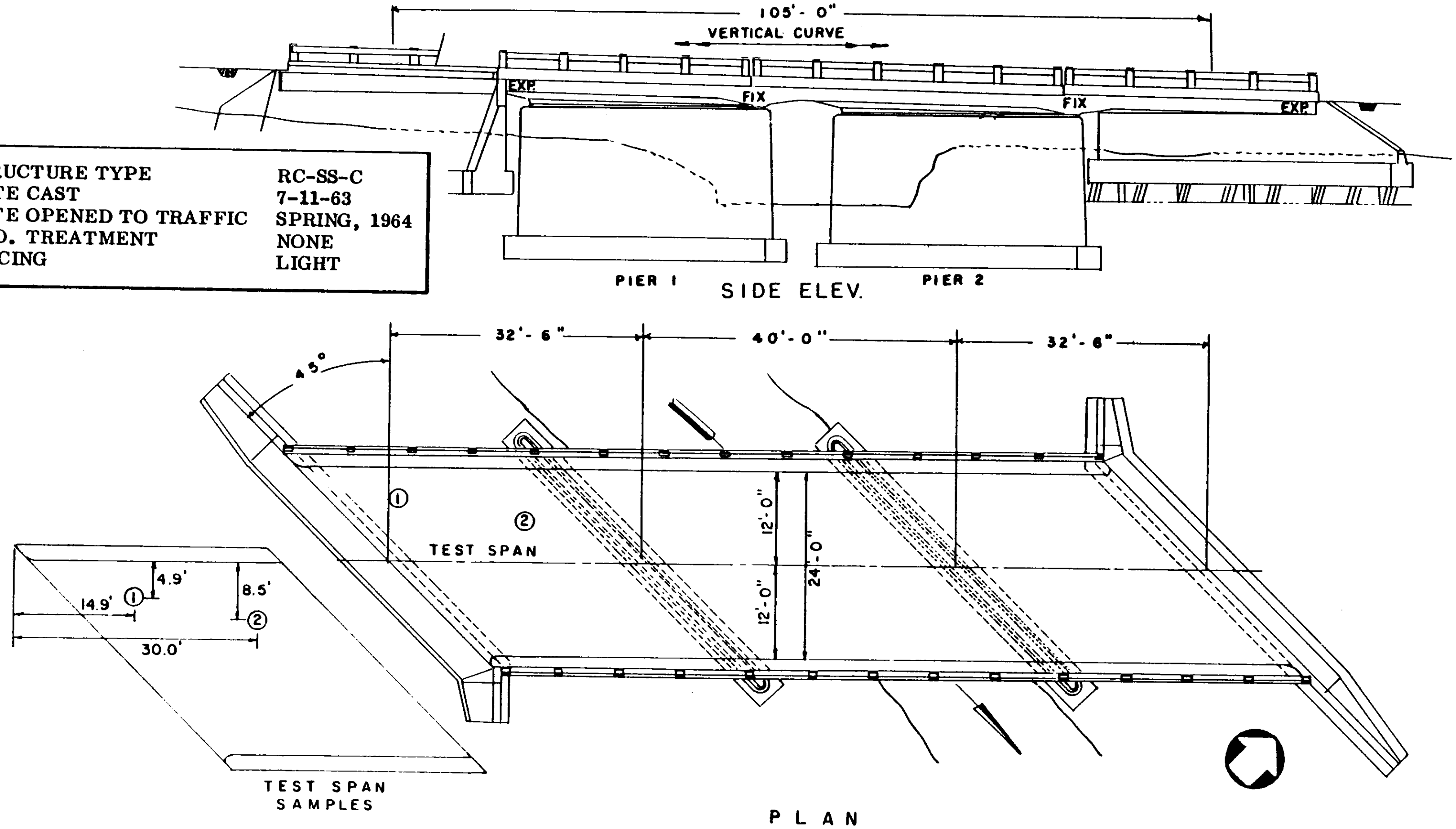
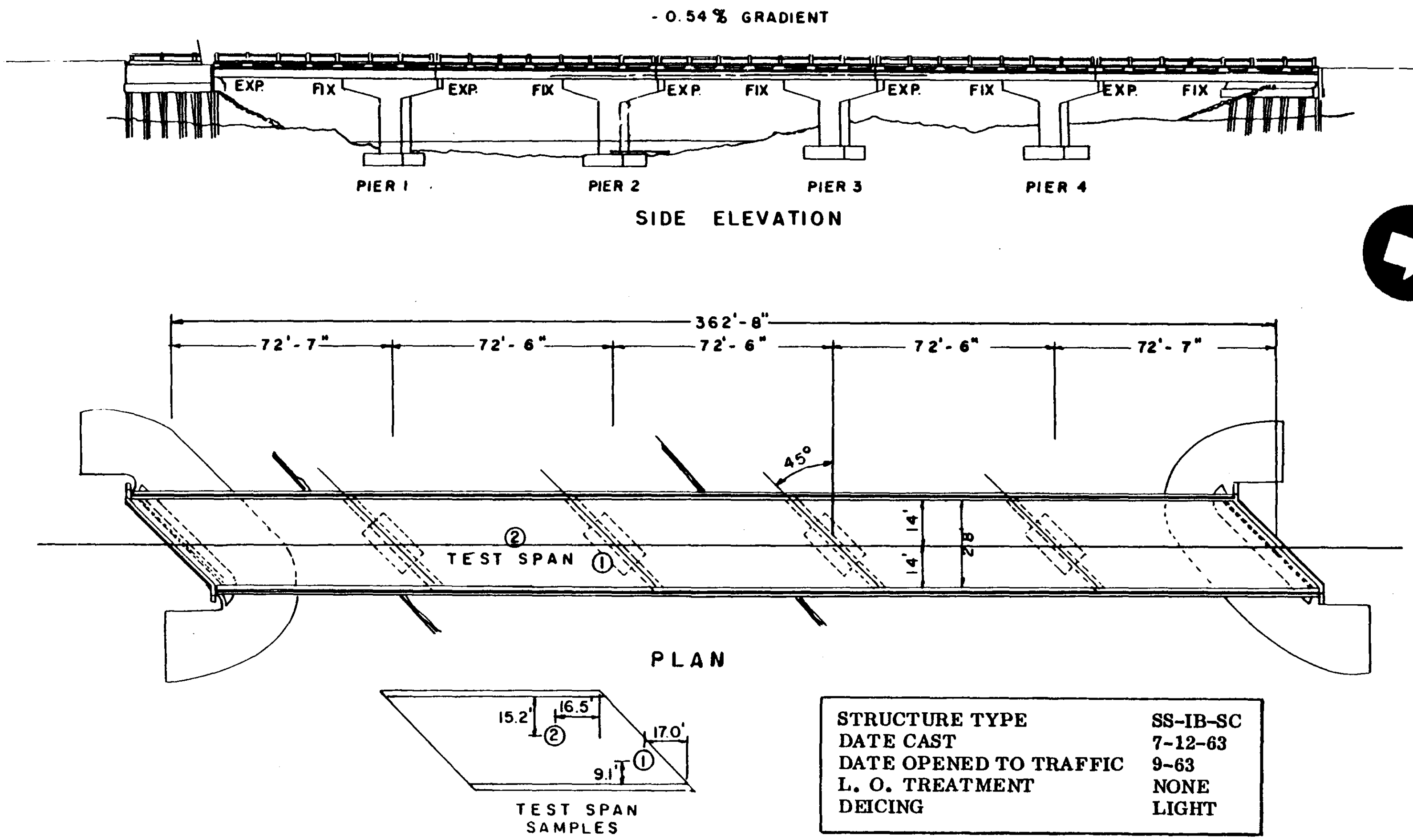
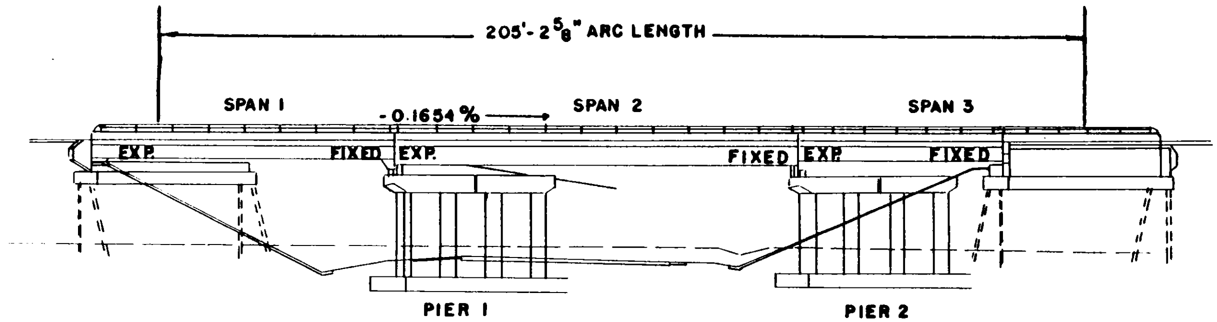


Figure A-10. Plan for bridge #10; also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB-SC
DATE CAST	7-12-63
DATE OPENED TO TRAFFIC	9-63
L. O. TREATMENT	NONE
DEICING	LIGHT

Figure A-11. Plan for bridge #11; also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB, DG-SC
DATE CAST	7-22-63
DATE OPENED TO TRAFFIC	10-63
L. O. TREATMENT	NONE
DEICING	HEAVY

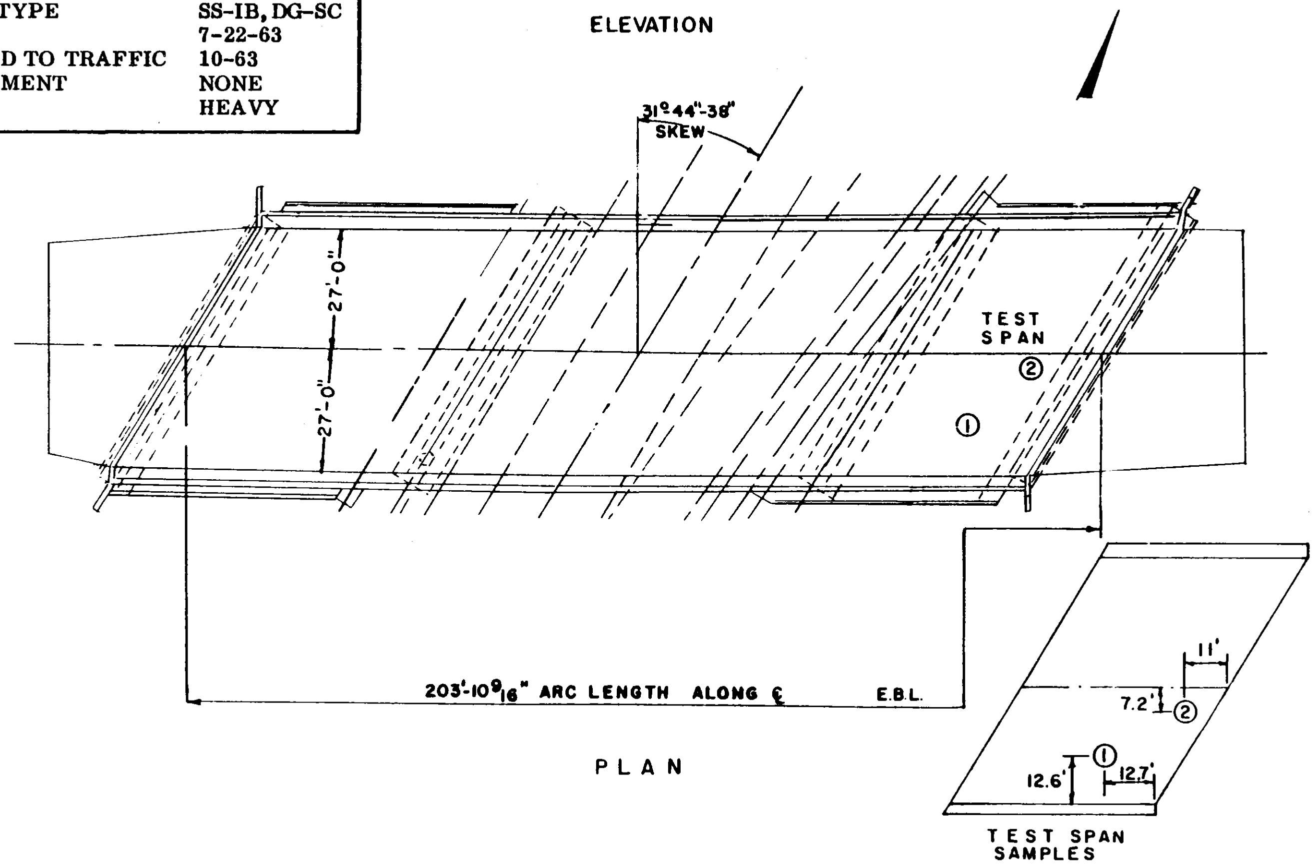
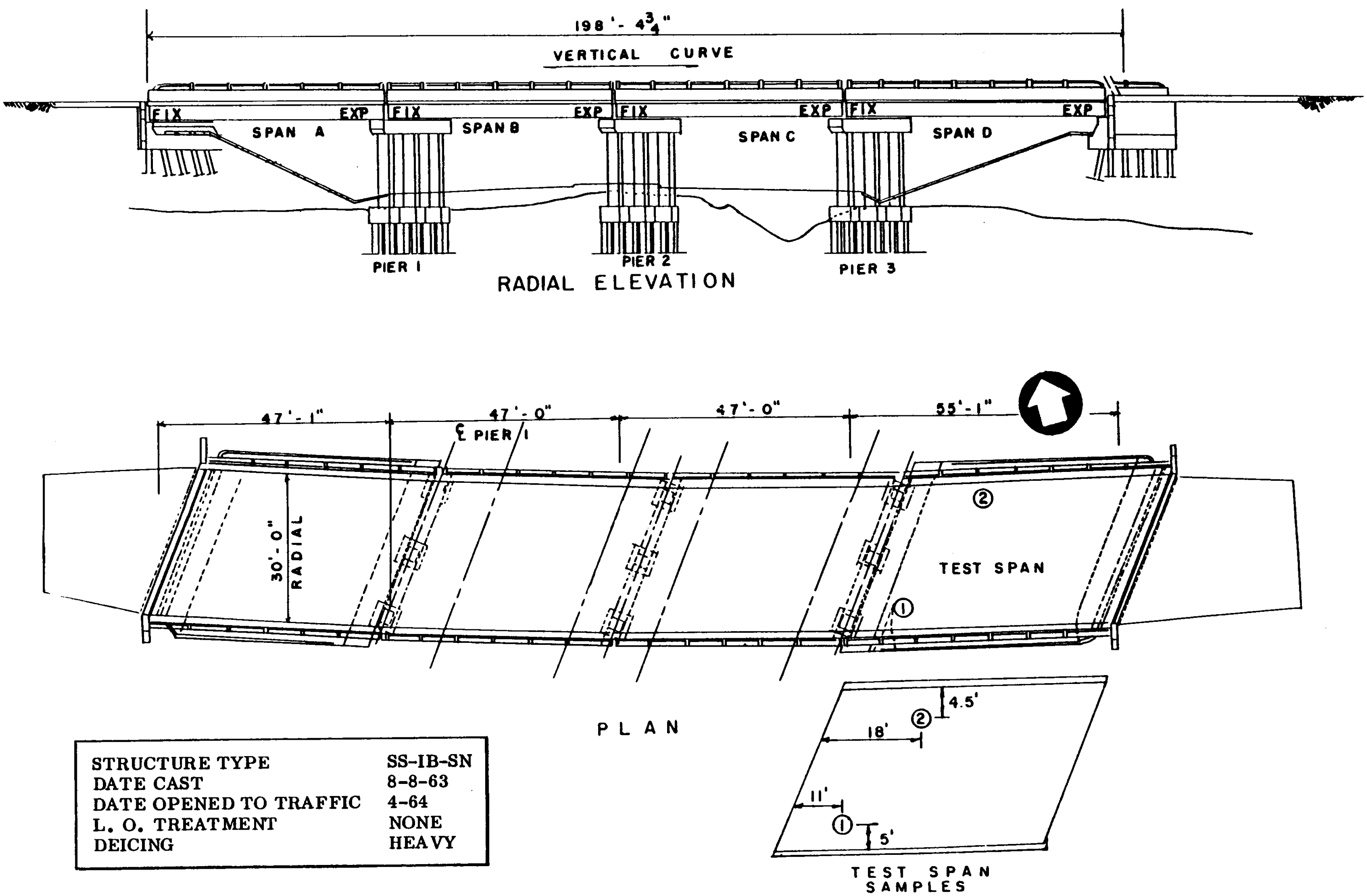


Figure A-12. Plan for bridge #12; also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB-SN
DATE CAST	8-8-63
DATE OPENED TO TRAFFIC	4-64
L. O. TREATMENT	NONE
DEICING	HEAVY

Figure A-13. **Plan for bridge #13; also shown are the locations of test samples.**

STRUCTURE TYPE	SS-IB-SC
DATE CAST	8-8-63
DATE OPENED TO TRAFFIC	12-16-65
L. O. TREATMENT	RESURFACE
DEICING	MEDIUM

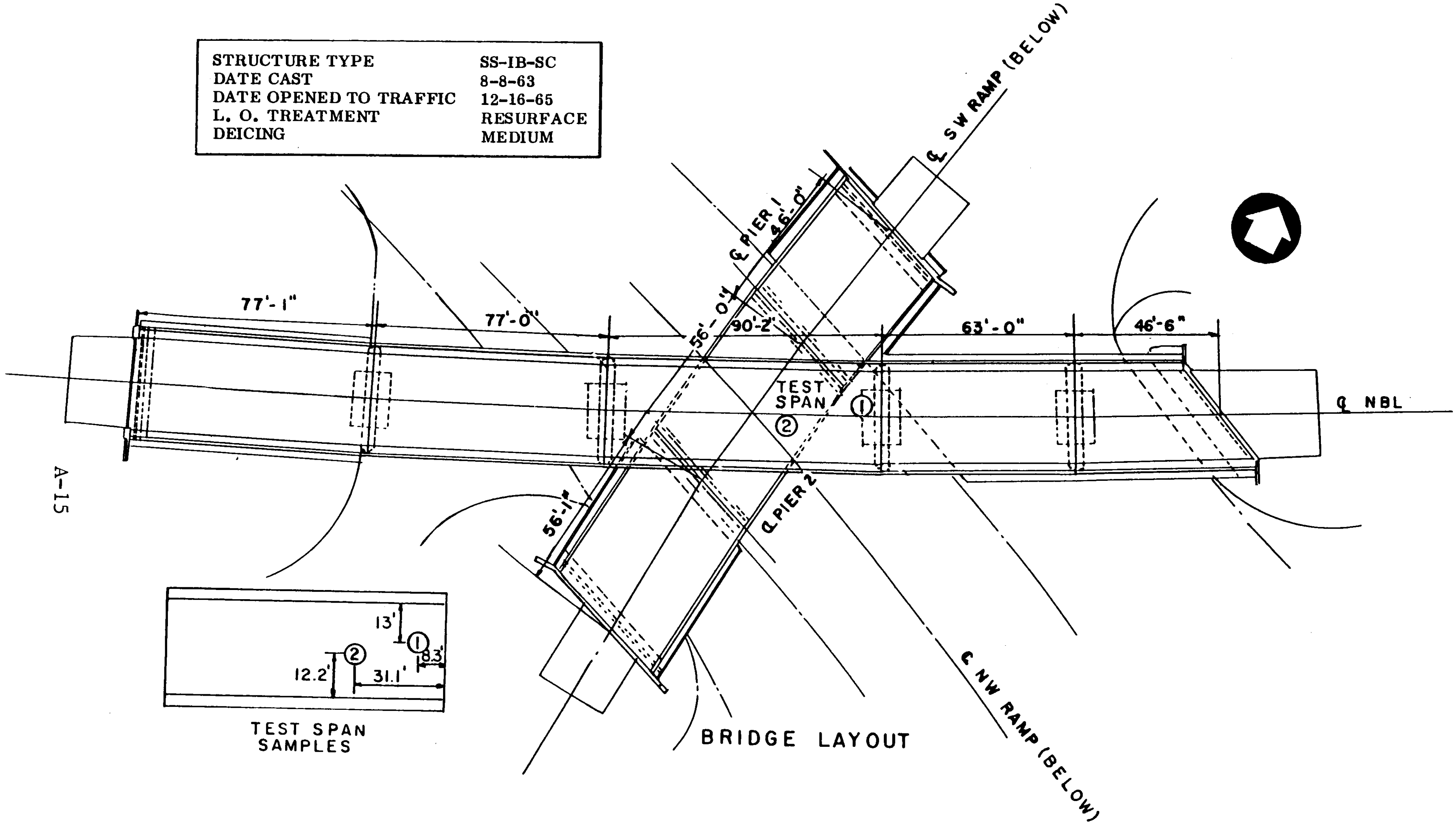
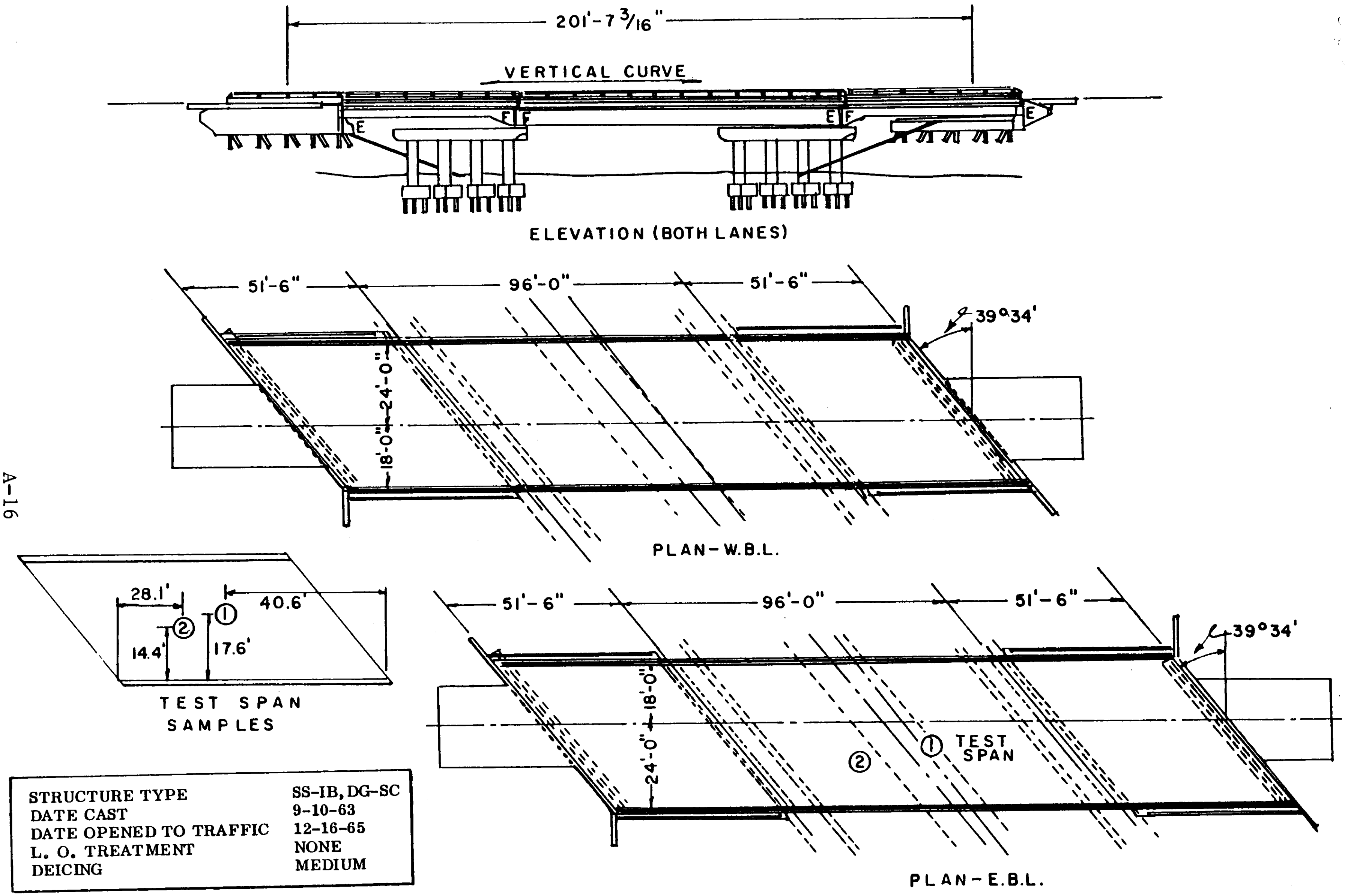


Figure A-14. Plan for bridge #14; also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB, DG-SC
DATE CAST	9-10-63
DATE OPENED TO TRAFFIC	12-16-65
L. O. TREATMENT	NONE
DEICING	MEDIUM

Figure A-15. Plan for bridge #15; also shown are the locations of test samples.

STRUCTURE TYPE	SS-IB-SC
DATE CAST	9-16-63
DATE OPENED TO TRAFFIC	12-63
L. O. TREATMENT	7-68
DEICING	HEAVY

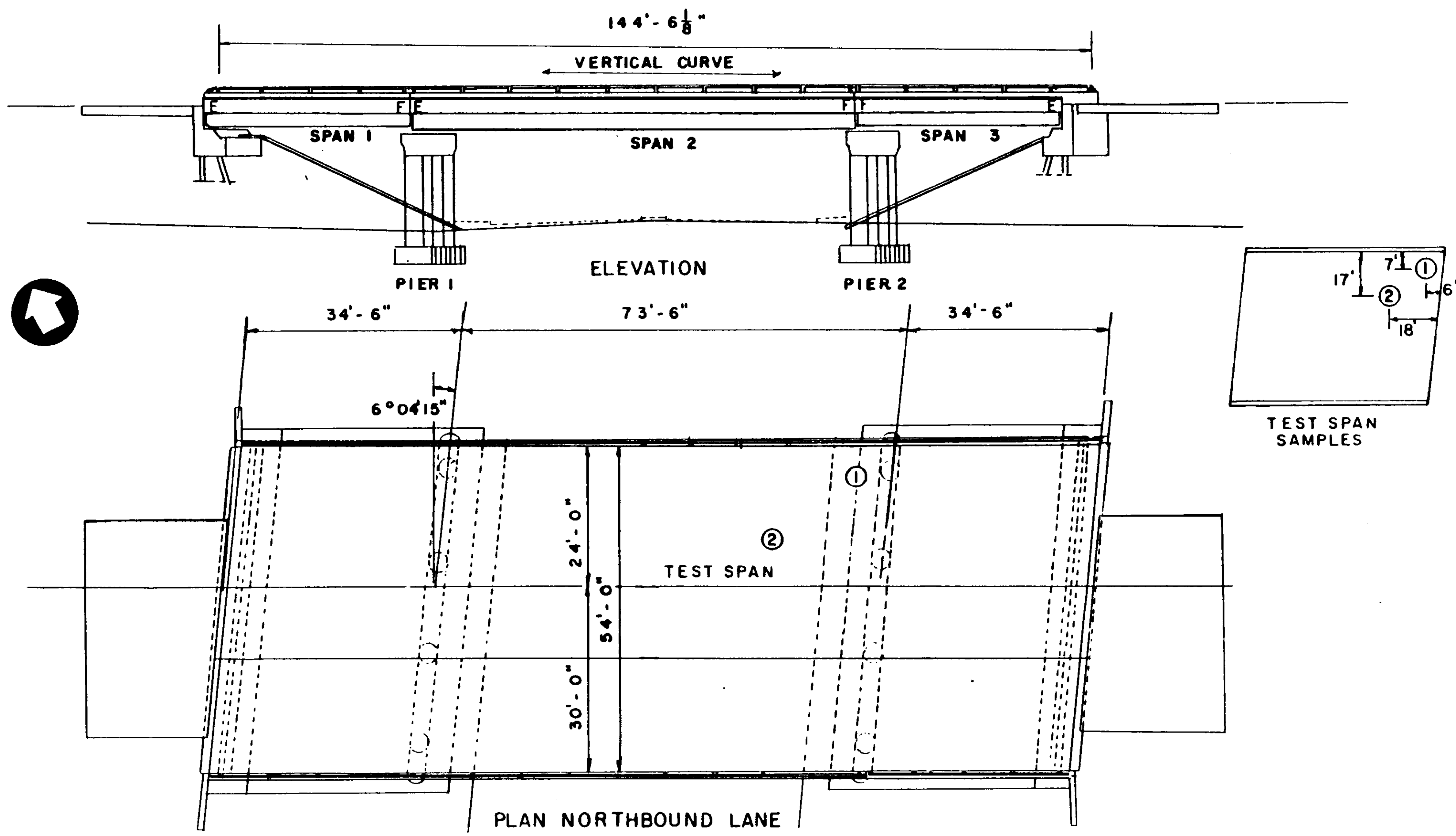
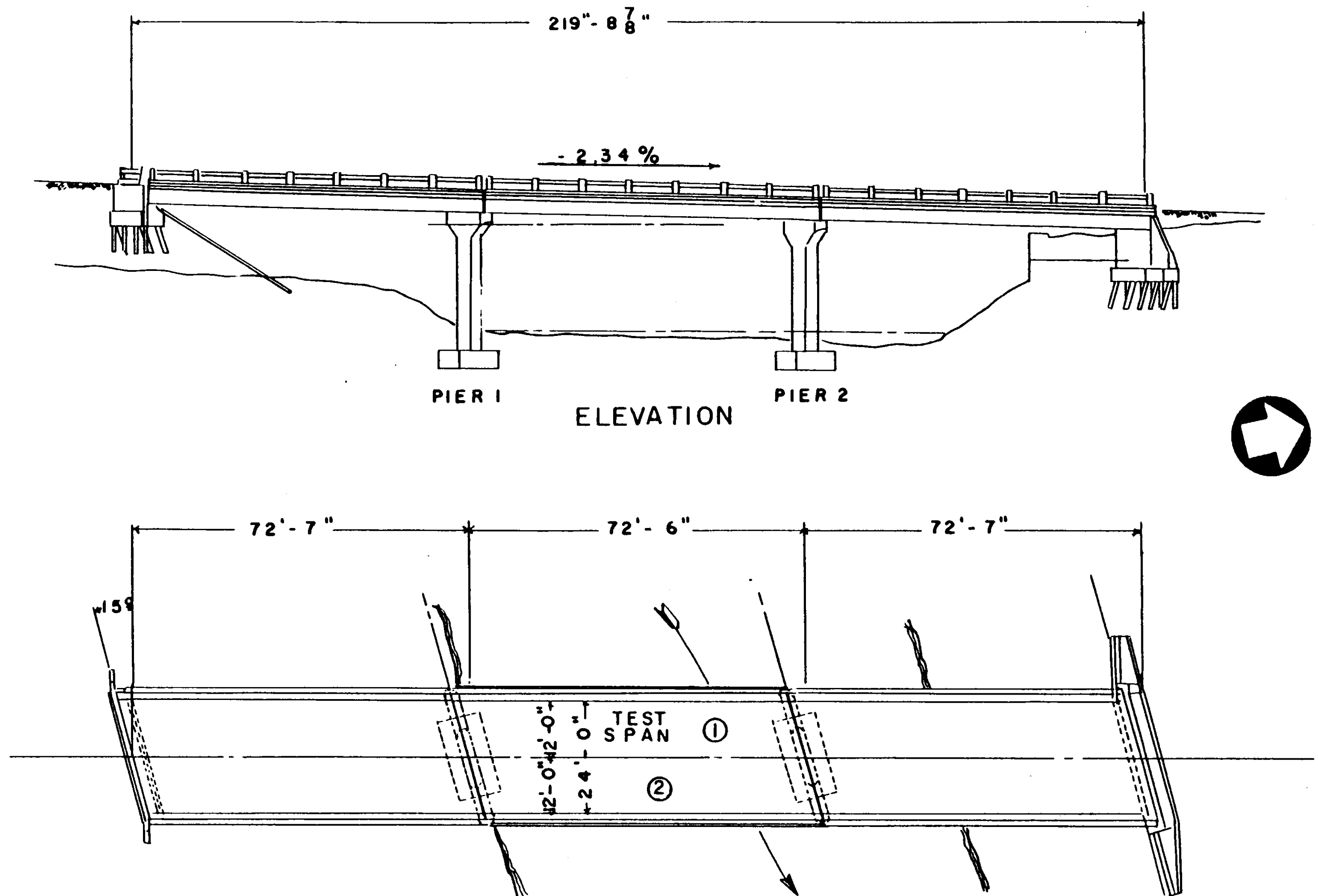


Figure A-16. Plan for bridge #16; also shown are the locations of test samples.



STRUCTURE TYPE	SS-IB-SC
DATE CAST	9-17-63
DATE OPENED TO TRAFFIC	10-63
L. O. TREATMENT	NONE
DEICING	NONE

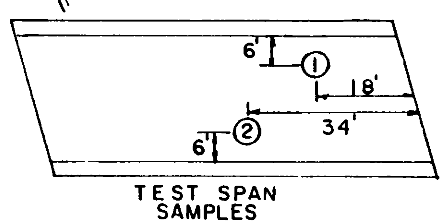


Figure A-17. **Plan for bridge #17; also shown are the locations of test samples.**

DATA SHEET FOR RANDOM BRIDGE SURVEY INSPECTION REPORT

HAYS CREEK AT BROWNSBURG

State VA. County ROCKBRIDGE Highway Nos. 252 Bridge No. CXL1-1

Year Built 1958 Deck: Uncovered ✓ Covered ______ Type of Cover NONE

Is detailed construction data available? ______

What type of deck repair or reconstruction has been done? NONE

Span No. 1 has been selected as the N (S) E W end of bridge. (Circle One)

****

AE

Classification of Deck Deterioration

Span No.		1	2	3	4	5	6	Remarks
Length-ft. Girder Type		57 SC-IB-SC	57	57	57			
Scaling	1	75%	90%	60%				1968 ADTC 315
	2	X	X	X	70%			
	3				X			
	4							
Cracking	1	M		H	L			
	2							
	3							
	4							
	5							
	6	M		H	L			
Rusting	1							
Surface Spall	1							
	2							
Joint Spall	1							
	2							
	3							
Pop-Out	1			FEW				

Comments: 2 LANES 4-57' S.B. SPANS (MSC-24-55½)
3rd SPAN - MIDDLE SECTION HAS HEAVY CRACKING TRANSVERSELY
4th SPAN - 20% HEAVY SCALING

Date of Inspection 6-17-70 Inspector NORTH - DAVIS District Office ______

Figure A-18. Survey form.
(See back for instructions.)

Instructions for Survey Form

Use one or more sheets for each bridge. Consider a dual bridge to be two individual bridges. A widened bridge is either dropped from the survey or inspected only for information on the old portion. Report any observed defects noted for each individual span.

Scaling is reported as an estimated percentage of the deck of an affected span for the average severity condition -- in box 1 for light scale; box 2, medium scale; box 3, heavy scale; and box 4, severe scale. An X is also placed in a box to designate the most severe scaling condition observed in the span. For example, in Figure A-18, 70 percent of the area of span 4 had an average scaling condition classified as medium scale, and heavy scale was encountered in portions of the scaled areas.

The six classifications of cracking -- box 1 for transverse; box 2, longitudinal; box 3, diagonal; box 4, pattern or map; box 5, "D"; and box 6, random -- are reported as being light, medium, or heavy (L, M, or H). Light cracking means widely spaced, fine cracks or only a few cracks in the span. Heavy cracking means closely spaced, wide (prominent) cracks, or many cracks in a span. For example, in Figure A-18, medium transverse cracking (box 1) is noted in span 1 of the bridge, heavy in span 3, and light in span 4.) Random cracking (box 6) of the same severity is found on the same spans. There is no visible longitudinal (box 2), diagonal (box 3), pattern (box 4), or "D" (box 5) cracking in any spans.

The presence of any rust stains on the deck surface is reported by an R in the box for the particular span.

Surface spalls are reported as small (box 1), or large (box 2). The number of spalls in each span are reported.

Joint spalls are reported by the estimated linear footage spalled along the joint. The spalls are classified according to the type of joint on which they occur along a metal expansion device (box 1); along a joint filled with sealing material (box 2); or along a construction joint or open joint (box 3).

Popouts are reported as being few (F) or many (M) in the judgement of the inspector.

APPENDIX B

CHARACTERISTICS OF STUDY BRIDGES

For the purposes of classifying structural types during the comprehensive national survey of bridge decks, the PCA and BPR adopted a three letter system which is utilized in this report and is described below.

Three groups of letters comprise the abbreviations: the first group designates the material (steel or concrete) in the main members; the second group describes the type (box girder, I-beam, truss, etc.) of main members; and the third group describes span type (simple, continuous, etc.). Abbreviations are as follows:

1. First group of letters:

   RC = Reinforced concrete
   PS = Prestressed concrete
   SS = Structural steel

2. Second group of letters:

   BG = Box girder
   DG = Deck girder
   IB = I-beam
   SS = Solid slab
   HS = Hollow slab
   TA = Trussed arch

3. Third group of letters:

   F = Rigid frame
   S = Simple spans
   C = Continuous spans
   SN = Simple spans, noncomposite
   CN = Continuous spans, noncomposite
   SC = Simple spans, composite
   CC = Continuous spans, composite

Examples:

RC-DG-C -- A reinforced concrete deck-girder (or T-beam) bridge having continuous spans.

SS-DG-CN -- A structural steel deck-girder bridge having continuous, noncomposite spans.

SS-IB-SN -- A structural steel I-beam bridge having simple, noncomposite spans.

RC-DG-F -- A reinforced concrete deck-girder, rigid frame bridge.

RC-HS-F -- A reinforced concrete hollow slab, rigid frame bridge.

SS-TA-C -- A structural steel trussed arch bridge having continuous spans.