COMPARISON OF PROPERTIES OF FRESH AND HARDENED CONCRETE IN BRIDGE DECKS

by

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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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PREFACE

This report is based upon data gathered on bridge decks designed and constructed under procedures which existed in 1963. These procedures, which were typical of and in some aspects more demanding than those in use in other states at that time, have subsequently been shown by research and experience to be less than adequate for the construction of bridge decks, which are subjected to a severe environment. Some defects have developed in the decks during seven years of service. These defects are largely explainable by the characteristics of the concrete or the procedures which were observed during placement. While premature deterioration is undesirable the purpose of this project, which was to relate performance to the observations made during construction, is better served by the fact that some defects have developed.

There is a natural tendency to blame individuals for failure when the perfection expected by most highway engineers is not achieved. The fixing of blame or condemning of individual failures is not the purpose of this report; the procedures followed in the construction of the bridge decks described were consistent with the "state of the art" as it existed nationally at the time. Throughout this report, emphasis is placed upon the changes that have been made in response to this and other research. Such responsiveness has resulted in better decks, from the standpoints of both riding quality and durability. Actually the period during which these bridges were constructed represents a time of transition in all aspects of bridge deck construction, and the results emphasize the soundness of the decisions which have been made, the emphases which have been placed, and the attention that has been directed to the concrete in bridge decks.

It is in this spirit that the project was initiated and pursued, and it is in this spirit that this report is written.

SUMMARY

A study was made on seventeen bridge decks constructed in 1963 under regular construction procedures.

The purpose of this project was:

(1) To compare important properties of concrete as freshly placed in randomly selected bridge decks with those after hardening of the concrete.

(2) To detect the influence, if any, of different screeding methods.

(3) To relate the performance of the decks to the observed properties of the fresh and hardened concrete.

The objectives and techniques were similar in many respects to those of projects reported elsewhere. The important difference was that the properties of the fresh concrete were more closely defined than in most other studies, which in many cases were forced to rely on project records, historical data, or observations on hardened concrete only.

The structures described in this report are representative of the last ones built under the old specifications, and are thus not representative of current practice. For this reason some of them incorporate concrete that should be susceptible to distress, according to current knowledge, and afford an excellent opportunity to evaluate this type of material.

From tests on freshly mixed and hardened concrete samples and observation of the performance during seven years the following conclusions are drawn:

1. When viewed against the perfection desired by the engineer, the performance of these decks has been disappointing or borderline. When viewed against the performance that would be expected from concrete with the properties observed, the performance has been better than might be expected.

2. The performance of the 17 structures in this study closely parallels the performance of a large sample of bridges included as a part of a nationwide study of bridge deck performance.

3. The primary cause of variable or borderline performance of concrete in bridge decks is variable or borderline fresh concrete. Many of the deficiencies have been overcome by changes in specifications and procedures instituted since the construction of the bridges included in this study.

4. Even with the use of elaborate mechanical equipment diligent attention must be given to the details of accepted practices of good concreting such as maintenance of low water cement ratios, adequate air contents, and prompt and thorough curing.

5. The agreement of properties such as unit weight and air content measured in both the freshly mixed and hardened concrete is acceptable for engineering purposes.

6. No influence of the screeding method on the properties of the concrete in place was found. However, the following indirect relationships may exist.

   (a) The average slump of concrete placed on jobs using mechanical screeding equipment was 2.8 while that for jobs utilizing hand methods was 3.7. To the extent that slump reflects water content, the use of mechanical screeds should result in a better quality concrete.

   (b) Of the four bridges screeded with the longitudinal screed, three have shown relatively serious deficiencies. Two have been resurfaced primarily because of deficient cover over the uppermost reinforcement. The third span has surface spalling, which also appears to be related to insufficient cover over reinforcement. Hopefully research nearing completion at the Research Council will shed light on this problem and suggest means for eliminating it.

7. Traffic volumes and design features seem to have had little influence in the adverse performance of the seventeen bridges in this study.

8. The only three bridges that are free from scaling are the only three bridges that contained an adequate entrained air voids system.

9. For the class of mixtures used in these decks, a minimum air content of 5 percent was found to be necessary in order to provide a void spacing factor of 0.0055", while air contents of 4 percent provided spacing factors below 0.0075".

10. The importance of the early application of curing was reflected in the scaling of several decks of apparently satisfactory concrete to which curing was applied very late.

11. Uncertainty exists as to the exact proportions of the components, especially water, in the concrete in these bridge decks. Procedures established since this project should improve this situation.

12. The influences of water-reducing admixtures on retarding the setting and reducing the water requirement were apparent in the samples from this project, as was the accelerating effect of high initial mixture temperature.

13. The data from three spans suggest that the cracking common to them might be explainable from high sand equivalents of the fine aggregates used.

14. Popouts were confined to structures using aggregates known to be susceptible to this type of defect.

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INTRODUCTION

Early in the nationwide emphasis on bridge deck studies, the Virginia Department of Highways instituted action to improve all aspects of its bridge deck construction. Initially, emphasis was placed upon smoothness. A committee composed of representatives from several of the Department's operating divisions and under the chairmanship of W. E. Winfrey began activity which culminated in the modification of various procedures and the issuance in 1965 of a 53-page guide for field engineers (VDH 1965).

Increased concern for scaling and/or cracking, coupled with the rapid development of mechanical screeding equipment, led to speculation about the combined and possibly beneficial effects of such equipment and other factors on the characteristics of concrete used in deck construction.

A proposal outlining the project described in this report was submitted in the spring of 1963 (Newlon 1963) and the field work was done during the period May 1963 to September 1963. Analyses, including periodic condition inspections of the decks, have been continued since that time. Interim results have been reported by various means within the Department including half-day discussions in each of the eight construction districts in 1964. These sessions were attended by about 300 field personnel. Many of the findings from this project have already been made use of in the Department's practices or specifications, or both. One report has been issued (Hilton, Newlon, Shelburne 1965). This report is the final one for the project.

Results from a resurvey of bridge decks originally inspected as a part of a nationwide survey of deck performance but later observed as a basis for interpreting the performance of the structures included in this research have been presented in a separate report (Davis, North, Newlon 1971).

PURPOSE AND SCOPE

The purpose of this project was:

(1) To compare important properties of concrete as freshly placed in randomly selected bridge decks with those after hardening of the concrete.

(2) To detect the influence, if any, of different screeding methods.

(3) To relate the performance of the decks to the observed properties of the fresh and hardened concrete.

The objectives and techniques were similar in many respects to those of projects reported elsewhere (NCHRP 1970). The important difference is that the properties of the fresh concrete were more closely defined than in most other studies, which in many cases were forced to rely on project records, historical data, or observations on hardened concrete only.

All the projects studied were of regular construction and were in progress when the research was initiated; thus they were not "experimental". Because of this constraint, the only tests used were those which would not interfere with normal operations.

Descriptions of the structures, procedures, and materials are given later. It is emphasized that the construction took place during a transition to specifications and procedures designed to eliminate problems that were becoming recognized in deck construction.

The structures described in this report are representative of the last ones under the old specifications, and are thus not representative of current practice. For this reason some of them incorporate concrete that should be susceptible to distress, according to current knowledge, and afford an excellent opportunity to evaluate this type of material.

CHARACTERISTICS OF BRIDGES STUDIED

General Features

For this study one deck span was selected from each of seventeen regular construction projects. The selection was such as to include several projects representing each of three types of mechanical screeding equipment; namely vibrating, longitudinal oscillating, and transverse oscillating. In addition, projects screeded manually were also included. A discussion of screeding methods and equipment is given in Appendix A. No distinction was considered necessary between longitudinal and transverse screeding except in the case of the oscillating screeds, which represented differences in design as well as direction of operation. Selection on this basis naturally resulted in bridge decks with a variety of aggregates, admixtures, and curing processes. Each of the seventeen decks, in effect, represented a unique combination of circumstances. Some of the more important characteristics of the decks and procedures are shown in Table I. Detailed sketches showing structural and other important features are shown in Appendix B. The structures and performance characteristics have been classified in accordance with the procedures developed by the BPR and PCA for their nationwide survey of bridge deck performance (BPR — PCA 1969).

TABLE I

IMPORTANT CHARACTERISTICS OF TEST BRIDGES

Job No.	Screeding equipment	Materials Coarse aggregate	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.

Materials

The requirements on concrete for bridge decks during the period 1938 – 1970 are shown in Table II. All of the concrete on these projects was supplied from ready-mix trucks under the requirements designated "1958" in this table. Measured values of these properties are presented in the text as necessary. It will be seen that the requirements in force during the period the bridges studied in this project were built were representative of those covering a long period of construction. Substantial upgrading occurred in 1966 as a result of several interrelated factors, including input from the observations from the research described in this report.

TABLE II

CONCRETE REQUIREMENTS 1938 — 1970

Date of specification	Cement content, sk./cy	Water-cement ratio, gal./sk.	Air content, %	Slump, in.	Maximum Aggregate Size, in.	20-day strength, psi	Aggregate quality requirements Maximum losses, % Los Angeles Abrasion coarse aggregate 100 rev.	500 rev.	Sulfate soundness loss** coarse aggregate / fine aggregate	Freezing & Thawing**
1938	$6\frac{1}{4}$	6	—	2–5	1	3,000	10	40	—	10(15)
1947	$6\frac{1}{4}$	6	—***	2–5	1	3,000	9	35	8(5) / 8(5)	5(15) / 5(15)
1954	$6\frac{1}{4}$	$5\frac{1}{2}$	3–6***	0–5	1	3,000	9	35	8(5) / 8(5)	5(15) / 5(15)
1958*	$6\frac{1}{4}$	$5\frac{1}{2}$	3–6	0–5	1	3,000	9	35	8(5) / 8(5)	5(15) / 8(15)
1966	$6\frac{3}{4}$	$5\frac{1}{4}$	$6\frac{1}{2} \pm 1\frac{1}{2}$	2–4	1	4,000	9	40	12(5) / 12(5)	5(20) / 5(20)
1970	$7\frac{1}{4}$	$5\frac{1}{4}$	$6\frac{1}{2} \pm 1\frac{1}{2}$	2–4	1	4,000	9	40	12(5) / 18(5)	5(20) / 8(20)

*These requirements were in force at the time the bridges observed in this project were constructed.

**Values in parentheses are specified numbers of cycles.

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

SAMPLING PROCEDURE

Two samples of concrete were obtained from each test span. By a random process the smallest identifiable unit (crane-bucket, buggy, etc.) was selected for sampling. For each sampling, pans were placed on the forms and the concrete deposited. The pans of fresh concrete were removed and the area precisely located for subsequent coring.

A number of standard tests were run on each sample to establish values for the important properties of the fresh concrete, including additional determinations of slump and air content. In all cases standard AASHO or ASTM procedures were used, and the tests run will be apparent from the results presented. The mortar fraction of the concrete was determined for each sample by washing the concrete over a No. 4 mesh sieve.

During placement and finishing, operations were carefully observed and particular attention was given to the age of the concrete and the times for each operation (i.e., vibration, screeding, texturing, etc.). After completion of the structures but before opening to traffic, four-inch diameter cores were removed from each of the two sample locations on 13 of the 17 test projects. These were evaluated petrographically, as will also be apparent from the results presented. All of the data collected by the construction inspector during routine control were also obtained and analyzed for comparative purposes.

Condition surveys of the structures have been made periodically. The characteristics of the sample area and the test span on each structure have been observed in detail. The condition of the remainder of the structure, including wheel guards, handrails, other decks, etc., has been ascertained in less detail for comparative purposes.

RESULTS

Characteristics of Fresh Concrete as Placed

Throughout the discussion of the results, it should be borne in mind that the concrete sampled had been previously subjected to control testing, including rejection of concrete judged to be deficient. The principal control indicators were a determination of slump and air content (by Chace indicator) on each load of ready-mixed concrete prior to placement in the deck. Thus, the variability reflected in the data is that for randomly selected samples from concrete that was, statistically speaking, "in control". Comparatively large variability would be expected, however, because of the use of only two samples per deck, the random location of the sample with respect to its position within the truck mixer, and the obvious differences expected among 17 projects.

Slump

A distribution of results from the slump tests for each of the 34 samples is shown in Figure 1. Two of the 34 samples (6 percent) exceeded the specified maximum of 5". This can be explained by the fact that the random sampling procedures sometimes required that the sample represent the early discharge rather than the conventional "average" concrete. Such sampling and testing variations are well established (Bloem and Gaynor 1970). The highest slump recorded was for a concrete containing a water-reducing and set-retarding admixture, and the water content as calculated was well below the specified maximum.

The slump varied considerably from job to job, as would be expected. The slump for the 20 samples from jobs using mechanical screeding equipment was 2.7", compared with 3.8" for the 14 samples from hand-screed projects. These results substantiate the fact that mechanical screeding commonly produces a stiffer concrete. The value of 2.7" conforms to the ACI recommendation that a maximum slump of 3 inches is satisfactory for mechanically consolidated concrete (ACI 309-1960).

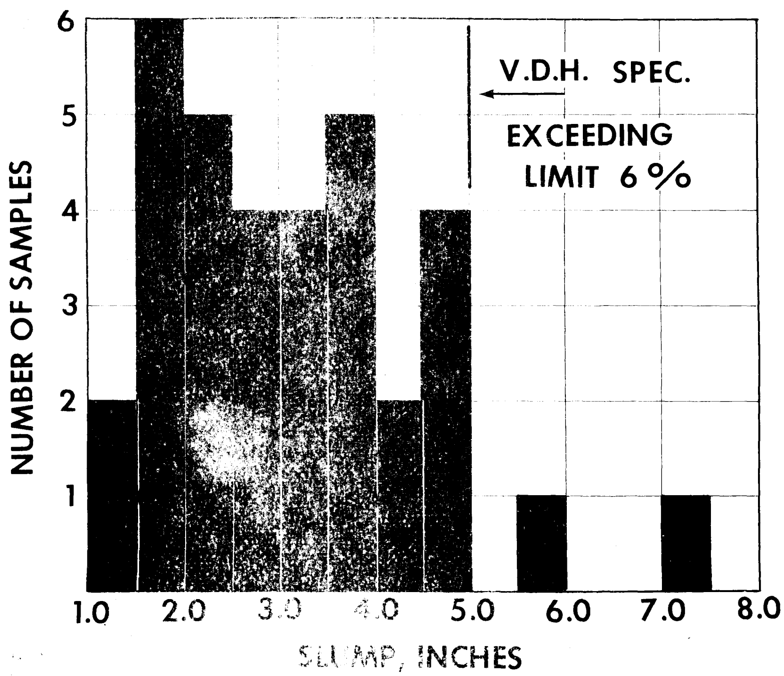

Figure 1. Distribution of slump measurements on fresh concrete.

The slump results in general reflect an appreciation of the importance of using low slump concrete on the part of construction inspectors and their diligence in measuring this characteristic.

Water-Cement Ratio

The significant influence of the water-cement ratio on all the important properties of concrete is generally given primary consideration in most concreting operations. Although slavish obedience to the "law" has been questioned (Gilkey 1961) and the significant contributions of other less appreciated factors demonstrated, the benefits of using the lowest possible amount of water cannot be questioned.

Unfortunately, there are no satisfactory test methods for determining directly either the amount of cement or the quantity of water, and thus resort must be made to calculations based on observed weights and/or correlations with strength results.

In this study the water-cement ratios were calculated conventionally from batch weights, corrected for aggregate moisture, and data from the wet-weight yield (ASTM C138). The values are subject to the considerable uncertainties which are always present when recorded data for weights and proportions are used rather than measurements of properties.

The water contents reported are shown in Figure 2. No special determinations were made. The values presented were calculated from the job inspectors' records of aggregate moisture and water added to the mixture. Two significant facts are indicated by these data. The first is that 7 of the 34 samples (22 percent) exceeded the specified limit of 5.5 gal./sack. Because of uncertainty in the moisture determinations, there is reason to suspect the validity of some of these data, particularly where high water contents coincide with very low slumps and in cases where strength data from job control tests are available. The pertinent point is that there is a considerable degree of uncertainty as to the water content of the concrete. A large portion of this uncertainty is related to the determinations of aggregate moisture.

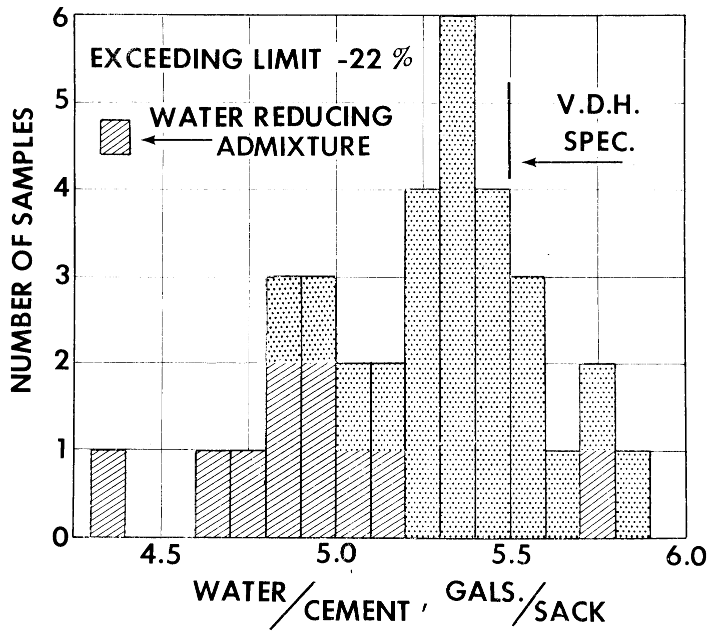

Figure 2. Distribution of water-cement ratios calculated for fresh concrete samples.

The second indication from these data is that they represent two separate statistical populations, dependent upon whether or not a water-reducing admixture was used. It will be seen that the use of the water-reducing admixture resulted in an average water reduction of about 0.5 gal./sack.

In Figure 3 the recorded water-cement ratios are compared with the "design" ratios for the project. In some cases the design values were below the maximum specification value shown earlier in Table II. About 30 percent of the samples contained more than the design water-cement ratio. Only one of the ten samples containing a water-reducing admixture exceeded the design water-cement ratio.

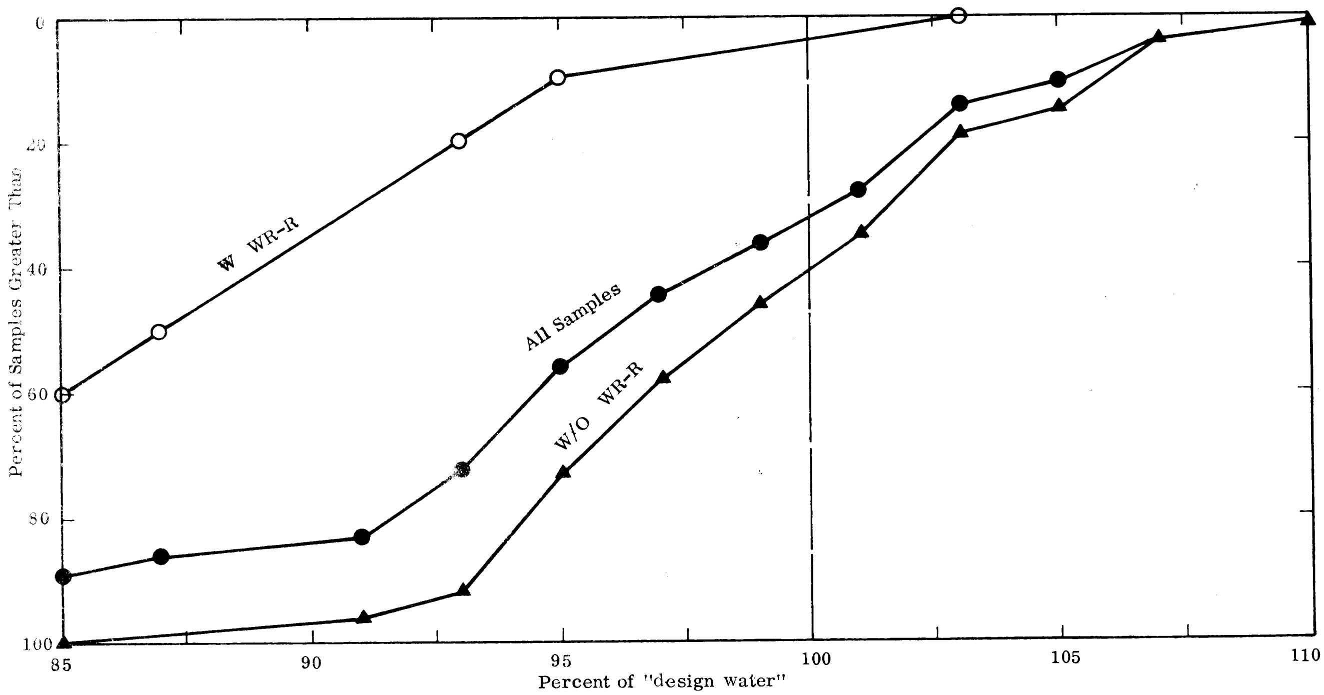

Figure 3. Frequency of occurrence of water contents in excess of that intended in the design.

At the time of these tests, the use of a water-reducing set-retarding admixture was optional, and the mixture proportions were not adjusted to compensate for the reduced water-cement ratio. Since 1965, its use for bridge deck concrete has been required. Also, at the time of the tests, there was some confusion in the field between the maximum water-cement ratio permitted by specifications and the maximum water content consistent with the job mixture. Beginning in 1964 (in fact, on several of the projects included in this study) the Department has required that a form including information on the "maximum design water" and the amount of water added to the load at the plant accompany each truckload of ready-mix concrete. These two modifications of procedure were intended to eliminate the situation reflected in Figure 2.

Air Contents

The distribution of air contents is illustrated in Figure 4. The air void system of the hardened concrete and its relationship to the air contents measured in the fresh concrete will be discussed later. These comparisons will show that in the majority of the cases studied, the poor air void characteristics are the result of a deficient amount of entrained air in the fresh concrete. Although the specification requirement at the time of sampling was lower than would be desirable, the problem was compounded by the tendency of the project inspectors to work to the lower limit rather than to the center or the upper limit. This fact is illustrated in Figure 4, which

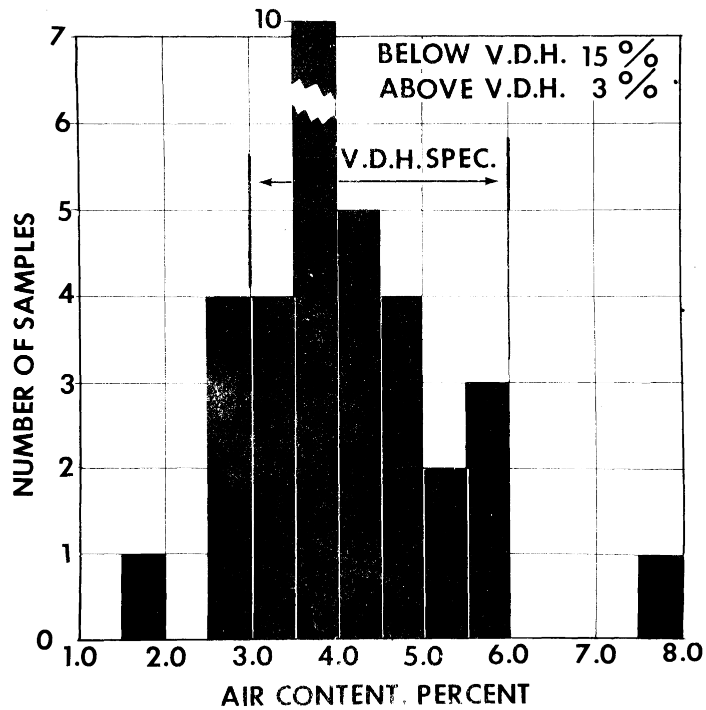

Figure 4. Distribution of air contents measured for fresh concrete.

presents the results of the air determination on the 34 samples from 17 projects. It will be noted that the distribution is skewed toward the low side of the range. Only one sample (3%) exceeds the upper limit while five (15%) are below the lower one. It is interesting to note that four of the five low samples were between 2.5 and 3.0 percent, which reflects the natural tendency to accept air contents which are only tenths of a percent below the required value. Twenty-four of the 34 samples (71%) are below the intended goal of the specification limits; namely, 4.5 percent. As will be described later, satisfactory spacing factors were obtained for the few samples in which the air contents in the fresh concretes were above about 4.5 percent, so that the quality of the concrete would have been considerably improved had the goal been the middle rather than the lower limit of the specification range. The tendency to work to the lower limit is understandable when one considers the premium placed upon attaining high strength concrete. Things that tend to decrease the strength (such as increasing the air content) are avoided.

Stiffening Rate and Mixture Temperature

The time of initial set as measured by penetration tests on mortar removed from the concrete (ASTM C403) is plotted in Figure 5 as a function of the temperature of the concrete at the time of deposition in the forms. Again, the samples containing a water-reducing retarder have been differentiated. Three important points should be noted. First, the temperature of 24 of the 34 samples (71%) was above 80° F.

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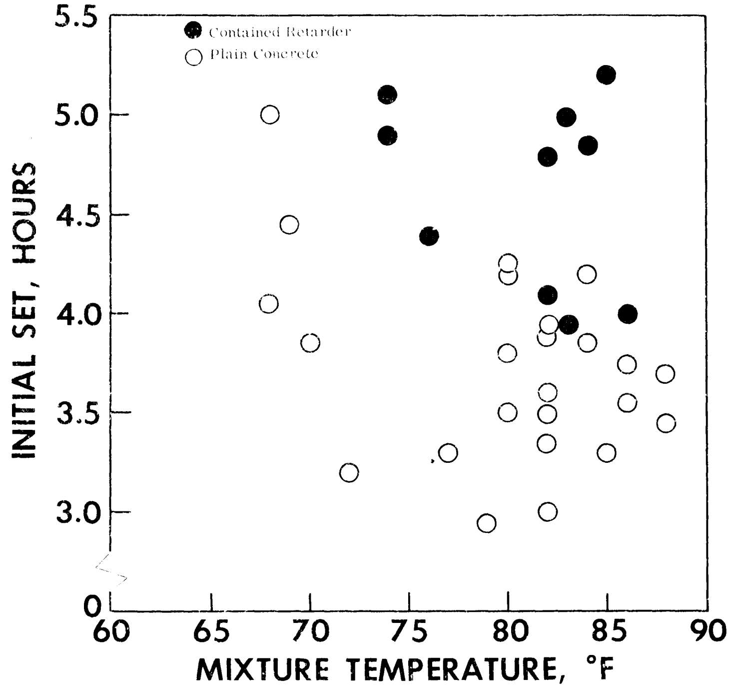

Figure 5. Relationship between initial set (ASTM C 403) and mixture temperature.

A recent report on the cooperative bridge deck study (Kansas, BPR, PCA 1965) cautions against the use of concrete with a temperature over 80°F. All of the samples were taken before noon when the lowest concrete temperatures for the day would be expected. The temperature of the concrete approached the upper limit of 90°F, which is often used as an absolute limit in hot weather concreting. Second, the expected trend toward more rapid set at higher temperatures is evident. The relationship is good, considering the many other variables in materials and environment involved in this field study. Lastly, the use of the water-reducing admixture extended the times of initial set by about 1.0 - 1.5 hours over those for the concretes without the admixtures. The stiffening rate data were observed to correlate well with the apparent behavior of the concrete in place. It should be emphasized that prolongation of the resistance to penetration is not synonymous with retention of slump.

Since these tests, the minimum cement content required for bridge decks has been increased to 6.75 sacks/c.y. Undoubtedly, with this increase has come higher initial mixture temperatures.

Yield

The yields determined for 33 of the 34 samples in accordance with ASTM C 138 are shown in Figure 6 as percentages of the intended yields. As noted later, the calculated proportions for a given job did not always yield exactly one cubic yard of concrete. The yields based upon the corrected calculated quantities are presented later. The basis for the values in Figure 6 would be what the normal contractor expected; i.e., 6, 7.5 yd., etc. Although no specification requirements are placed on yield, instructions suggest several actions which should be taken... "should the actual yield of concrete at the completion of any day's run vary by more than ± 2 percent from the theoretical yield". As noted in Figure 6, 6 of the 33 samples were outside the desirable range. The general trend was for yields to be slightly below the theoretical value. In view of the fact that minor variations in the testing procedure reflect noticeably in the results, corrective action should not be taken on the basis of a measurement on a single sample. For two of the test spans results from both samples were outside the desired range while for two other test spans only one of the two samples was outside of the range. In any event, the tendency was for lower than expected yields. Possible causes for the variations in yield are discussed in the next section.

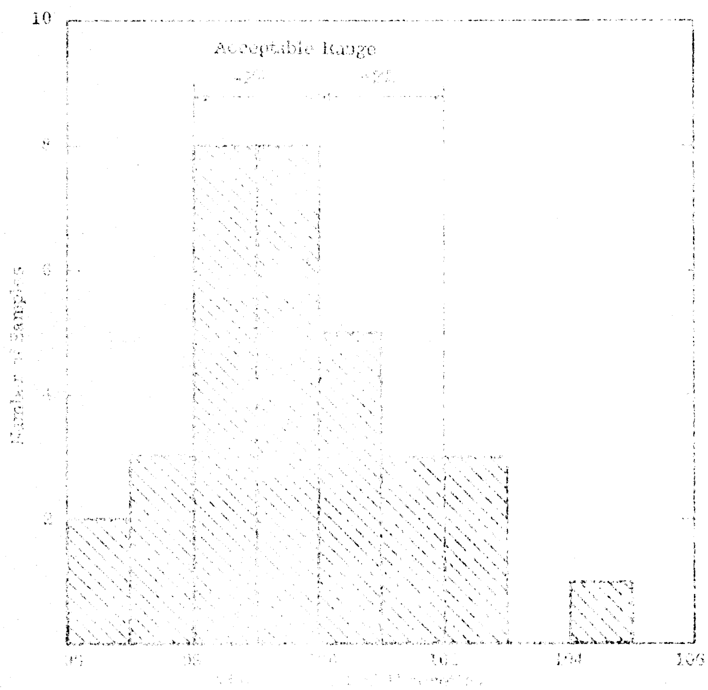

Figure 6. Distribution of yields calculated from fresh concrete.

Variation of Components

In addition to the measurement of air contents and yields and the recordation of water added at various points, the amount of aggregate coarser than the #4 screen was determined by washout tests. No direct measurements were made of fine aggregate, cement, or water. These were monitored at the appropriate points by observation of the scales and gages.

Thus, for each batch there were three groups of data on mix composition. The first were the batch or "design" weights determined for the Class A-4 concrete to be used throughout the projects. At present, concrete proportioning is based upon use of ACI 613; however, at the time of this construction, each construction district was arriving at these quantities in a slightly different fashion, but all were using proportioning principles tempered by local experience. These variations in proportioning methods resulted in the fact that when calculated on a common basis, not all of the "design" weights would produce exactly one cubic yard of concrete.

The second set of weights were those recorded as batch weights for the sampled batches. These varied from the "design" values primarily in the amount of water added and from inability to produce exactly the intended air content.

The third set of data was developed from the field measurements of yield, coarse aggregate, and air contents. These measurements, supplemented by the recorded values for cement, fine aggregate, and water, were used to calculate what are designated "measured" quantities.

Volumetric quantities were calculated from the three sets of data with the intent of reconciling them with the yield data and other observations made during construction and with subsequent performance. An example of the measured and computed values is given in Figure 7. In the example shown, the quantities reportedly batched were less than those of the design, and the yield measured was less than that batched. The volume of coarse aggregate measured was, however, greater than that reportedly batched.

The interactions are so complex and the recorded information, particularly that on water contents, is of sufficient uncertainty that detailed comparisons are not justified. Several points are, however, of some interest.

Shown in the first column of Table III are the measured values for coarse aggregate contents expressed as percentages by weight of the batch quantities recorded for the coarse aggregate. For 17 of the 32 samples,* the measured quantities were greater than those reportedly batched, while in 15 cases the measured quantities were less. In 9 cases the differences exceeded five percent.

*Note: Samples 16-1 and 16-2 were intermixed during testing and thus were unidentifiable so they were excluded from the analysis.

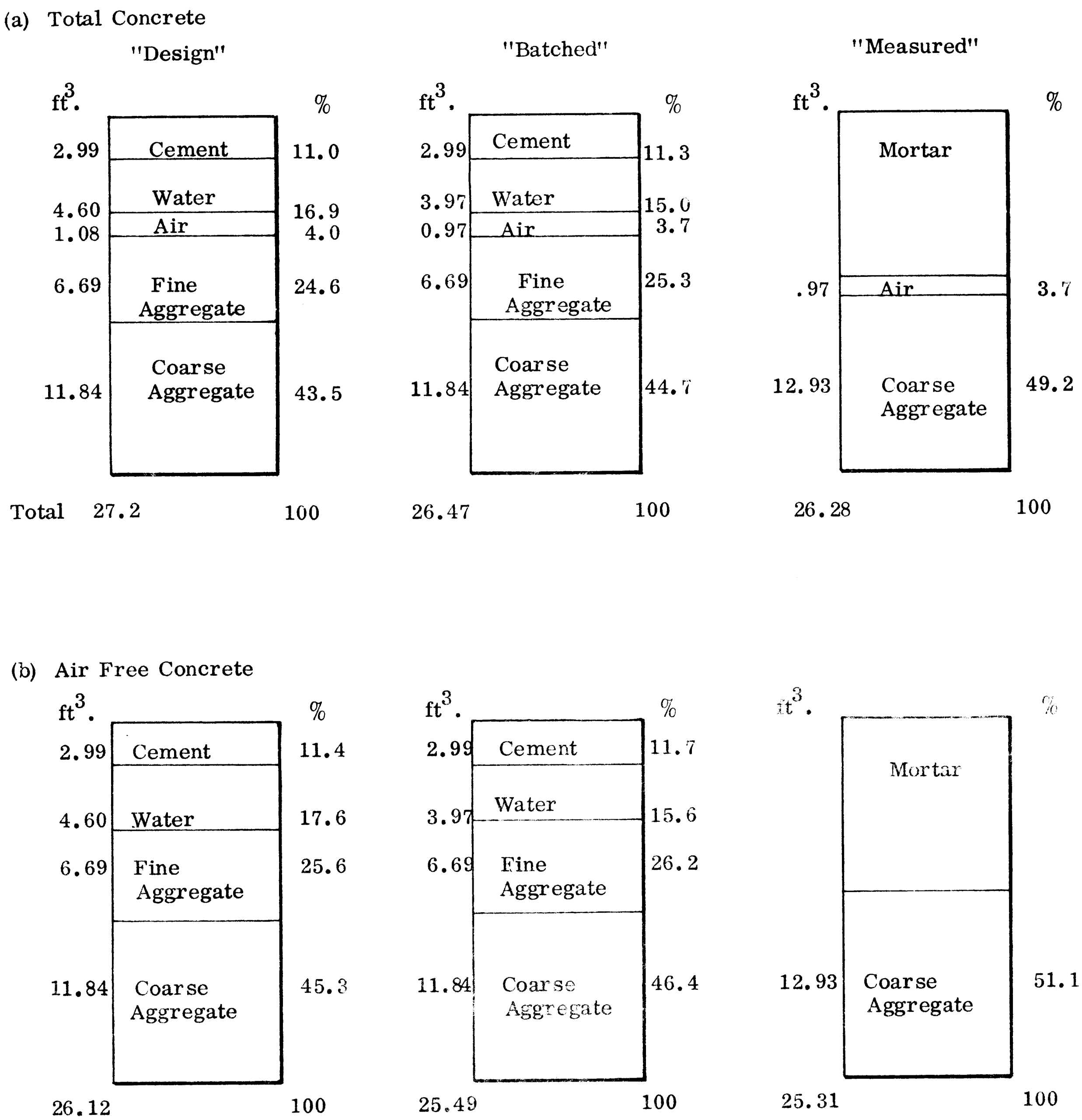

Figure 7. Volumetric quantities calculated from reported and measured data.

TABLE III

MEASURED COARSE AGGREGATE CONTENTS

Sample	Measured coarse aggregate content compared with batched quantity, percent by weight	The difference between measured and batched coarse aggregate (air free basis), percent by volume of concrete
1-1	109.9	+4.7
1-2	102.4	+1.1
2-1	94.9	-2.8
2-2	95.3	-2.9
3-1	96.5	-1.9
3-2	98.8	-1.1
4-1	98.6	-0.6
4-2	100.8	-1.4
5-1	97.3	-1.9
5-2	95.8	-1.9
6-1	107.0	+3.5
6-2	104.0	-1.8
7-1	100.1	-0.3
7-2	99.0	-1.1
8-1	95.3	-2.8
8-2	90.5	- .47
9-1	94.1	-3.6
9-2	109.1	+2.4
10-1	108.6	+3.3
10-2	105.3	+1.6
11-1	101.3	+1.5
11-2	98.1	-0.9
12-1	97.1	+0.6
12-2	100.9	+1.6
13-1	106.5	+3.0
13-2	102.0	+1.2
14-1	98.6	+0.3
14-2	101.0	+1.4
15-1	101.7	+1.4
15-2	101.9	+1.4
17-1	101.7	+1.1
17-2	100.0	+1.5

Shown in the second column are differences between the measured volumes of coarse aggregate calculated on a volumetric basis as a percentage of the total volume of air free concrete and the volumes reportedly batched. A plus value indicates coarse aggregate in excess of the batched quantity and also represents the portion by which the mortar is deficient. No further breakdown of components within the mortar phase can be made since these components were not measured and the total difference can result from any combination of variations of cement, fine aggregate, or water.

The distribution of absolute volumes calculated on an air free basis from the three sets of data is presented in Figure 8. As shown in Figure 8(a), 9 of the 32 samples represented situations in which the differences between the designed quantities and the measured quantities exceeded ± 2 percent. As seen from Figure 8(b), in 4 of the 9 cases (1-1, 1-2, 7-1, and 7-2) the differences are consistent with a failure to batch the weight intended in the design. These were cases in which the use of less water for concrete containing a water-reducing admixture was not compensated for by the use of other solid materials. Some inferences can be made regarding the 5 cases of high yield (3-2, 8-1, 8-2, 9-1, and 9-2). For samples 3-2, 8-1, 8-2, and 9-1, the results indicate excess mortar. While no exact component can be singled out, it is more likely that the excessive component is water. Thus, one would suspect that the water content of these samples is higher than recorded. Sample 9-2 contained an excess of coarse aggregate. It was noted during placement that this batch was "harsh". Excess coarse aggregate coupled with the probability of a water content like that of sample 9-1 explains the very high yield. The large differences between samples for bridges 1, 6, 8, 9, 10, 11 and 17 suggest that the batching and mixing procedures on these projects might have been deficient.

It should be noted that several of the samples had large deviations in coarse aggregate/mortar ratios which were not reflected in the various yield calculations. These deviations apparently were compensated for within the mortar.

If the yield of the concrete, including entrained air, is considered, the results are explainable by variations in entrained air,which has the most pronounced effect of any component on yield. A one percent variation in air content affects the yield in the same amount. As was shown earlier in Figure 4, variations of air content were common and those obtained were generally below the design values. The distribution of yields calculated to include air are shown in Figure 9 in the same format as were shown on an air free basis in Figure 8. In this case, the yields are more than 2 percent below the design feature in 5 of the 32 cases, while in 7 of the 32 cases they are more than 2 percent above. Several of the samples occupy the same position in both Figures 8 and 9. In other cases the air contents either compensated for other deficiencies or aggravated differences which were apparent on an air free basis. Samples 2-1, 2-2, 11-1, and 14-1 had higher air contents than expected. Samples 4-1 and 15-2 were lower. In other cases, the differences in yields reflect causes other than air contents.

In summary, the yield data provide some qualitative insights into compositional variations, but, in general, they are not sufficiently definitive for quantitative judgments.

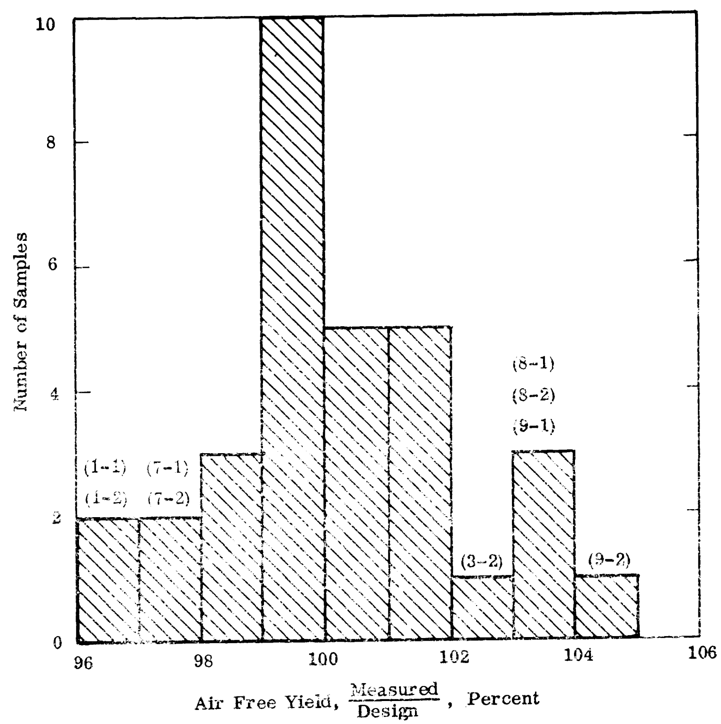

Figure 8(a). Distribution of measured vs. design yields calculated on an air free basis. Numbers indicate samples outside of ±2 percent.

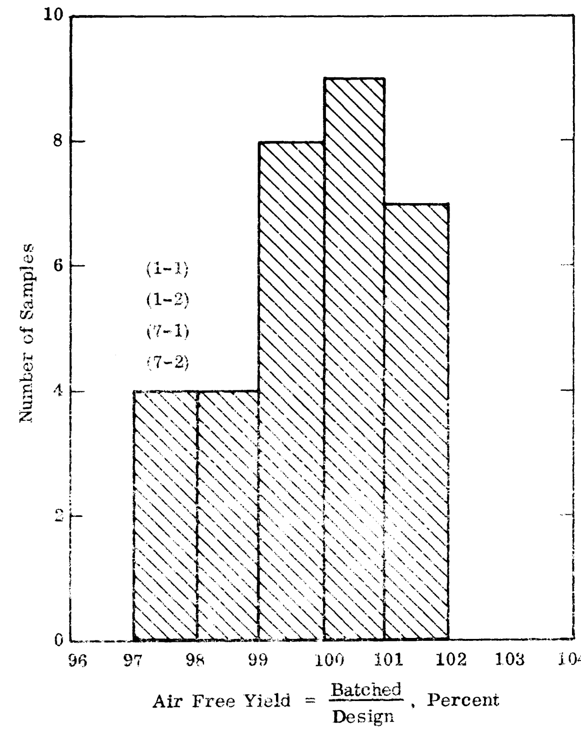

Figure 8(b). Distribution of batched vs. design yields calculated on an air free basis. Numbers indicate samples outside of ±2 percent.

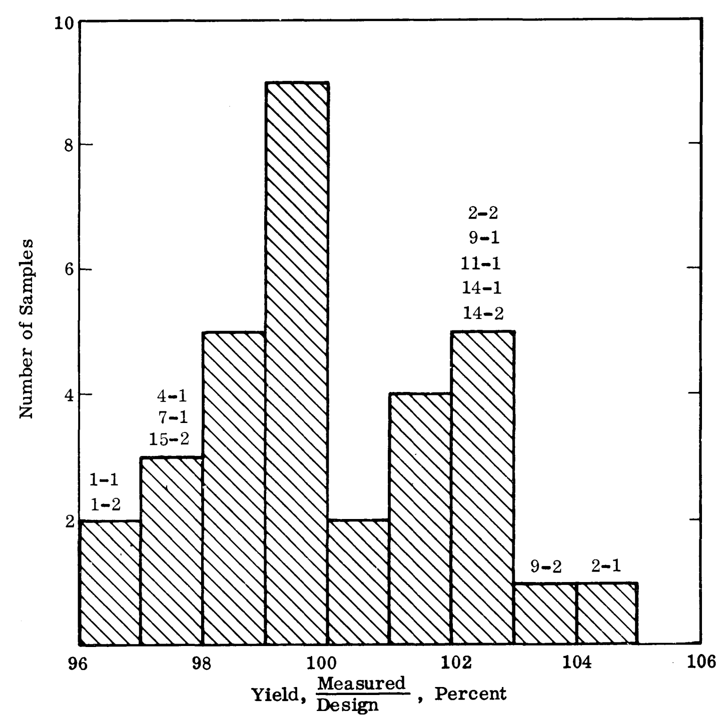

Figure 9(a). Distribution of measured vs. design yield calculated to include entrained air. Numbers indicate samples outside of ±2 percent.

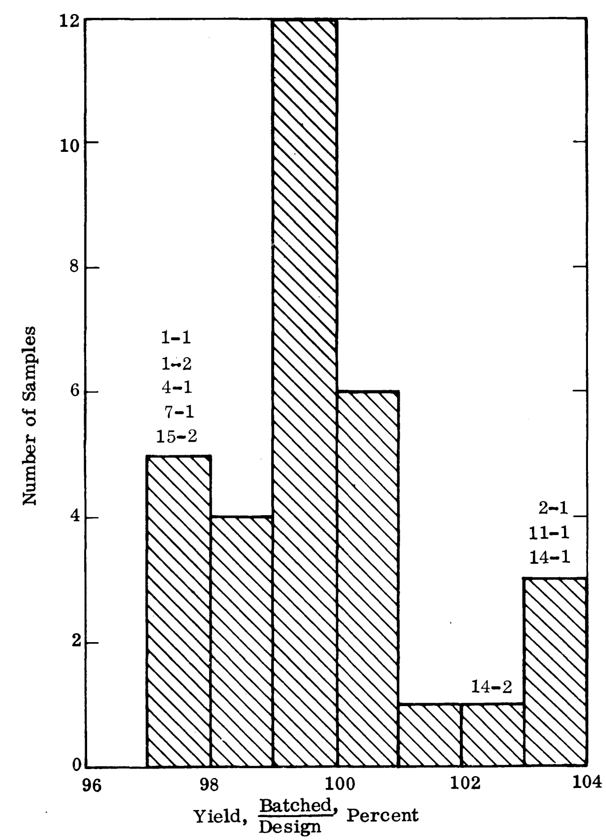

Figure 9(b). Distribution of batched vs. design yields calculated to include entrained air. Numbers indicate samples outside of ±2 percent.

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Construction Operations

Construction procedures were closely observed and recorded with the intention of relating performance characteristics of the decks or compositional variations of the concrete to these procedures. Individual relationships will be discussed in connection with the performance data, but several general observations are of interest.

The timing of various construction procedures was observed in order to detect relationships to the stiffening rate data from concrete tests. For this purpose the steps in the construction sequence were classified as:

(1) Mixing

(2) Placement (spreading, consolidating)

(3) Screeding (leveling, finishing, floating, etc.)

(4) Application of final texture (brooming or belting)

(5) Curing

As would be expected, the operations were quite variable. In all cases the screeding and finishing operations were completed at least one hour before initial set was indicated. Concrete is usually considered ready for application of the final texture at that time when the "sheen" disappears from the surface. Except for instances where construction operations precluded it, an attempt was made to observe this guide.

In general, the texture was placed earliest on concrete mechanically screeded transversely. The time of texturing with longitudinal mechanical screeding varied considerably but was generally earlier than with hand methods. The average difference between the time of initial set and time of texturing for the four categories of equipment studied are given in Table IV.

TABLE IV

RELATIONSHIP OF DELAY BETWEEN TEXTURING AND INITIAL SET

Type Screeding	Number of Samples	Average Delay Between Texturing and Initial Set, Hr.*
Mechanical Oscillating (Transverse)	8	-1.3
Mechanical Vibratory (Transverse)	4	-1.2
Mechanical Oscillating (Longitudinal)	8	-0.5
Hand	14	-0.4

* Negative sign indicates texturing before initial set.

The state of maturity necessary for application of curing is essentially the same as that stated for texturing. A measure of curing efficiency is, therefore, the elapsed time between texturing and the application of the curing medium. A distribution of these times is shown in Figure 10. It will be seen that for 12 of the 34 samples (41%) the delay between texturing and curing exceeded one hour. It will also be noted that membrane curing was usually applied more rapidly than covering mediums such as burlap, paper, polyethylene, etc. The notable exceptions are the two samples from a single job (#11) in which 5 hours elapsed between brooming and the application of the curing membrane. The sprayed curing membranes, though applied earlier, were not always adequate as seen in Figure 11. In the case of coverings, the tendency was to wait until a large segment of the work was completed before application, as shown in Figure 12. No systematic influence of screeding equipment on curing was noted, largely because of the strong influence of the curing medium. From the data shown in Table IV it would follow that more rapid application of curing would be possible with the use of transverse mechanical screeds.

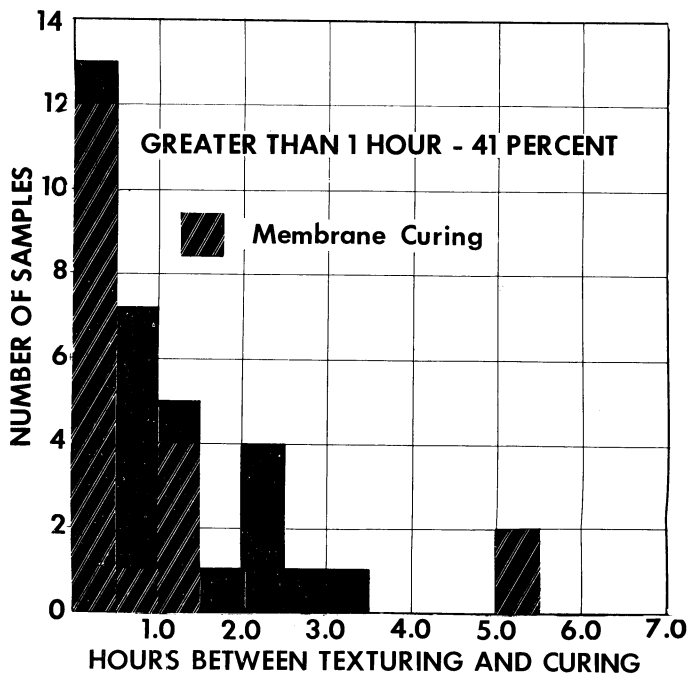

Figure 10. Curing efficiency as indicated by delay between application of texture and curing medium.

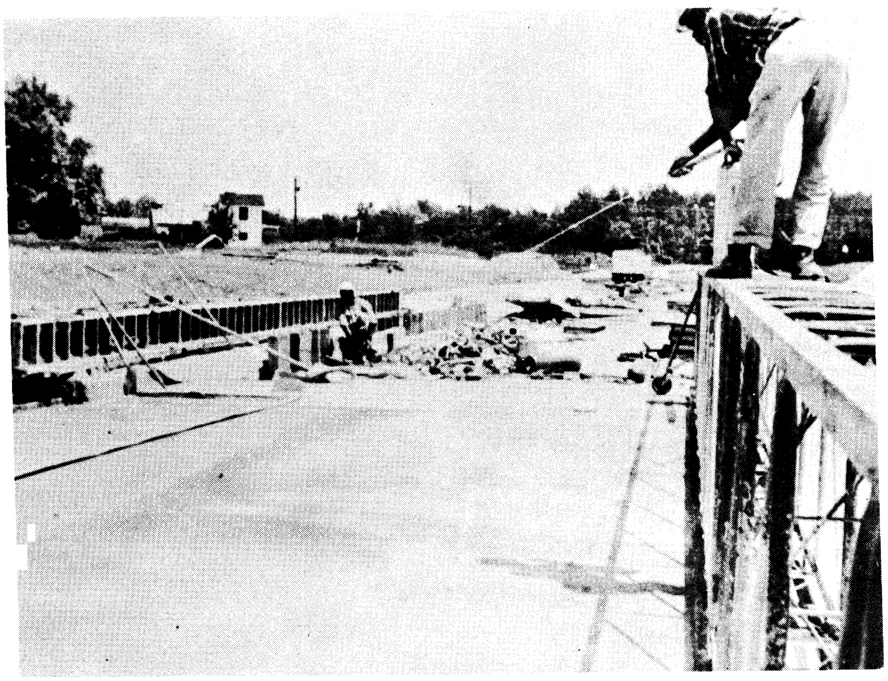

Figure 11. Improper application of sprayed on curing compound. Lack of a work bridge contributed to this situation.

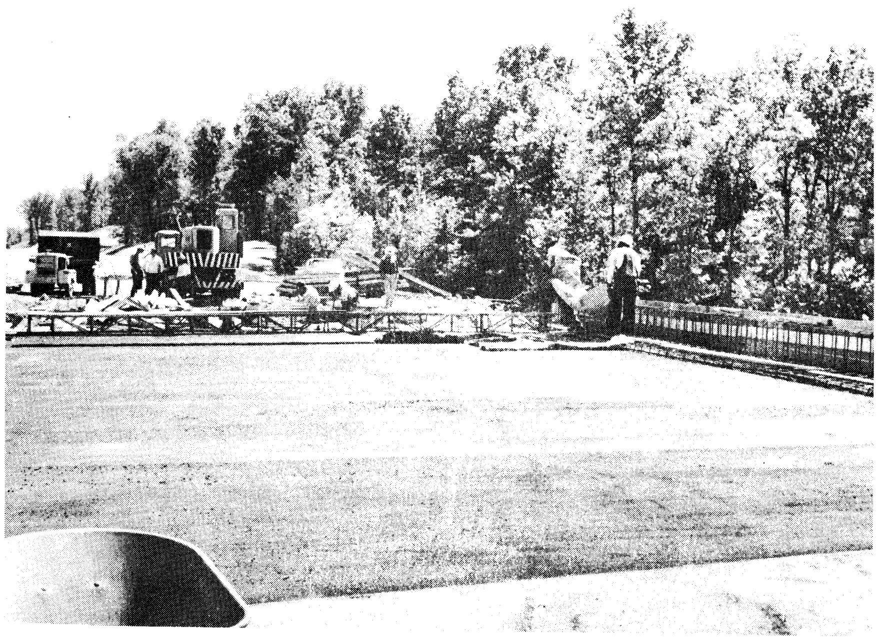

Figure 12. Delay of applying curing paper until a large portion of the slab was completed.

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Properties of Hardened Concrete and Comparison with Those of Fresh Concrete

Absorption and Unit Weights

Before the study structures were opened to traffic, one four-inch diameter core was removed from each of the two locations at which the concrete was tested in the fresh state. Twenty-six cores were taken from 13 of the 17 study structures. A steel locator was used to insure prevention in the core of the uppermost reinforcement. The primary purpose for removing the cores was for microscopic determination of important characteristics at various levels within the core. Before preparing the specimens for examination, however, the general condition, density, and absorption were determined for the core as a whole.

Macroscopic observation of the overall condition of the cores disclosed no cracking or other unusual defects. The density and absorption were determined for each core based upon drying at 100°C for 48 hours, followed by immersion in water for 24 hours. The unit weights converted from the bulk specific gravity (SSD) values ranged from 141.1 to 158.5 pounds per cubic foot. The relationship between the unit weights determined from the hardened concrete cores and those measured for the fresh concrete is given in Figure 13. The standard error of estimate from the regression equation was 2.0 pounds per cubic foot, and the correlation coefficient was 0.91. These indicate a reasonably good agreement between this gross property as measured from the sampling under different conditions (fresh and hardened) and lends confidence to the remaining comparisons of the properties between the fresh and hardened concrete.

The distribution of absorption values for the 34 cores is given in Figure 14. Absorption is not necessarily a very meaningful measure of frost resistance of concrete since it does not reflect the difference in type among the voids and is only vaguely related to permeability, which reflects the degree to which the voids are connected. Lea and Desch (1956) indicate that the absorption of most good concretes should fall below 10 percent. The values for the cores are well below this value, ranging from 1.83 to 7.86 percent. The absorption values of the coarse aggregate after 24 hours of gravity saturation ranged between 0.10 and 0.65 percent. An interesting observation is the generally lower absorption of concretes which contained a water-reducing retarder. The lower absorption likely reflects the expected reduction in absorption accompanying reduced water-cement ratios.

A plot of absorption for the hardened concrete as a function of the reported water-cement ratio is shown in Figure 15. This relationship is less clear than that of Figure 14 but still reasonably good when consideration is made for the uncertainty of the reported water-cement ratios and the effects of variables other than water-cement ratio on the absorption. It is significant that cores from jobs 8 and 9 show higher absorptions than would be expected from the reported water-cement ratios. This is consistent with earlier arguments which suggest that the actual water (i.e., increased absorption with increased water-cement ratios) contents of these samples were higher than reported. A similar case can be made for the anomalous behavior of samples 1-1 and 1-2. Considering paris of cores from the same deck, the absorption increases or remains the same with increased water cement ratio in all but a single case (Job #1).

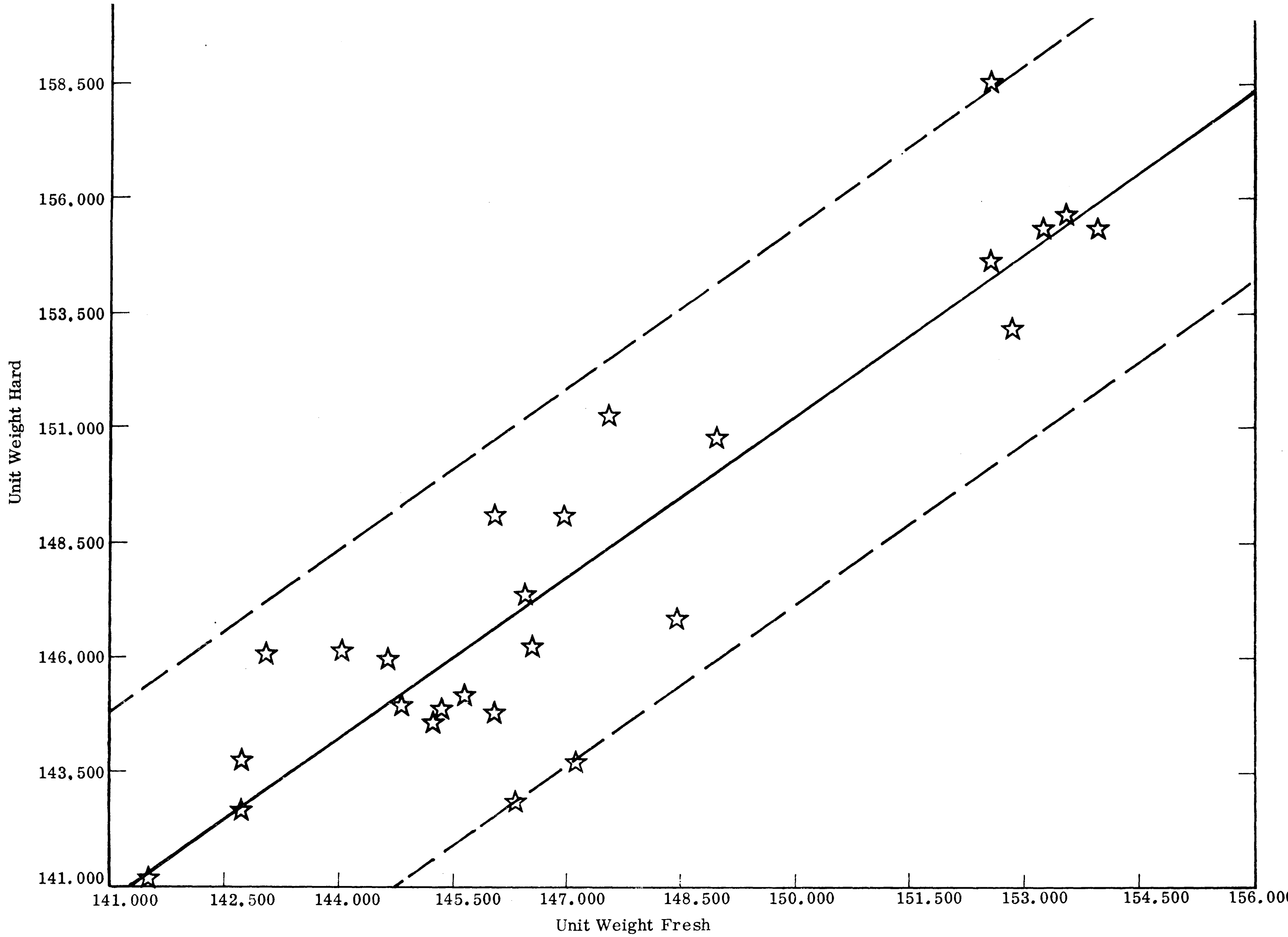

Figure 13. Relationship between unit weights measured in fresh and hardened concrete.

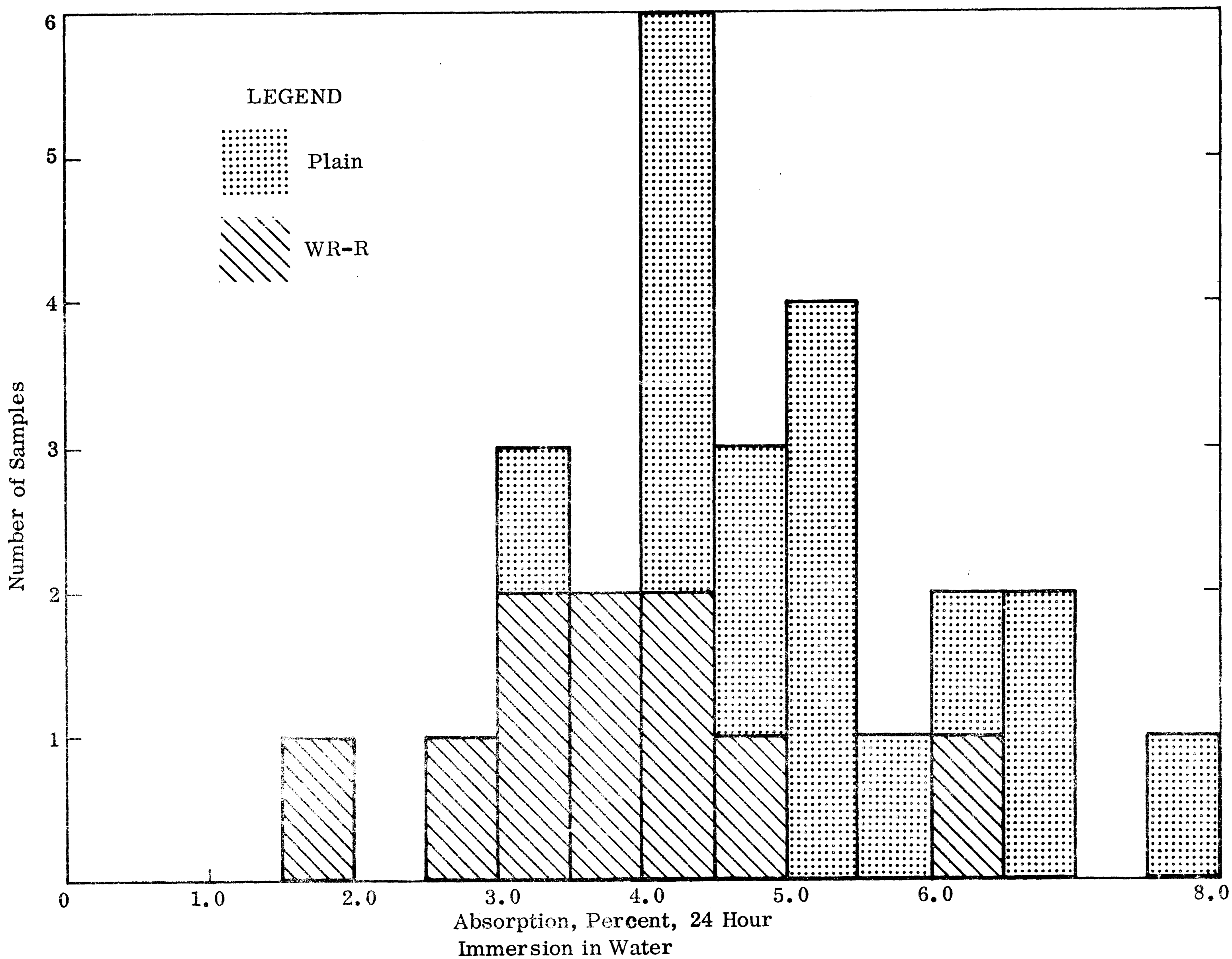

Figure 14. Distribution of 24-hour gravity absorption results.

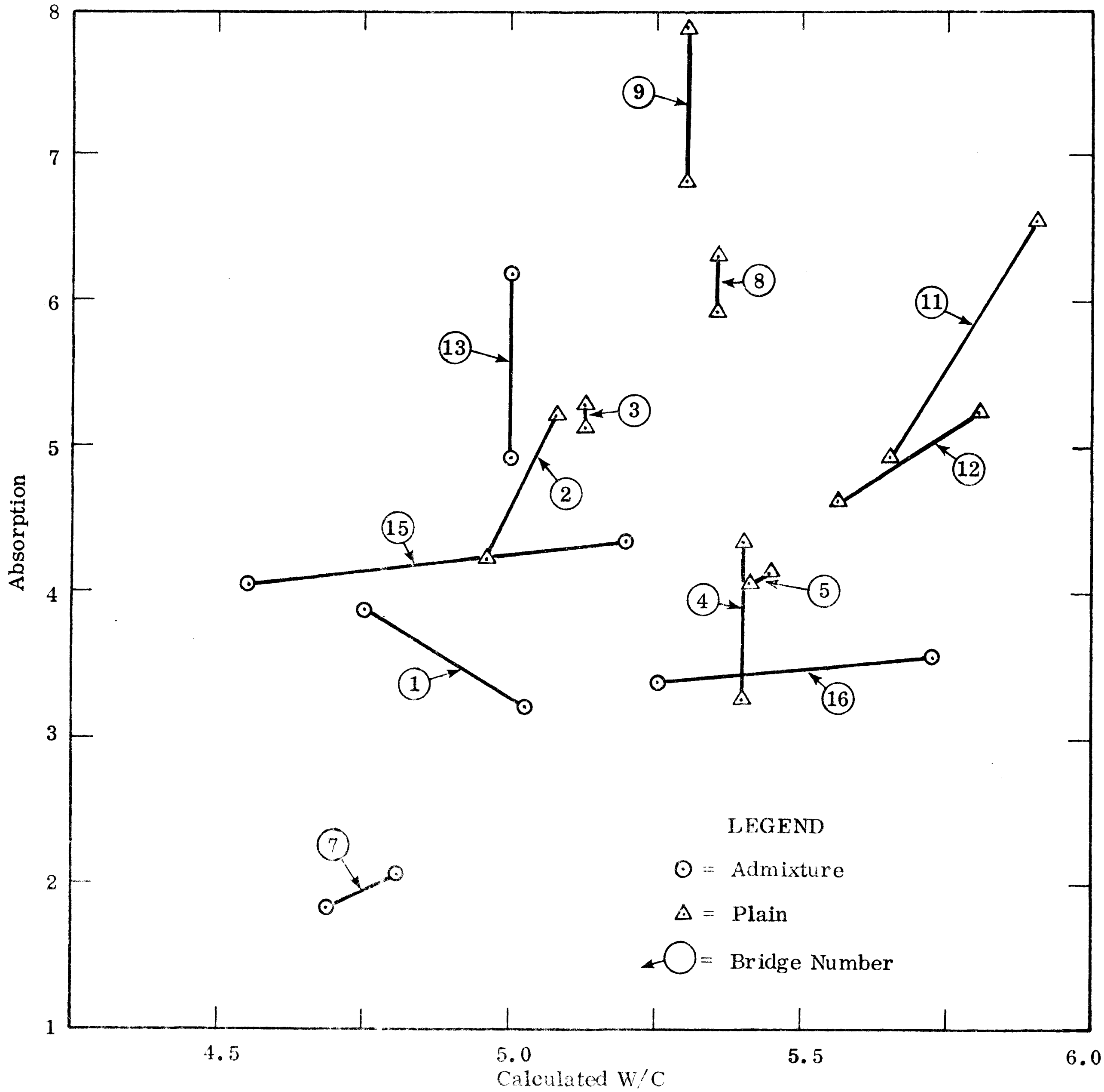

Figure 15. Relationship between measured absorption and calculated water-cement ratio.

Air Void Contents

The 26 four-inch cores were studied in detail using microscopic linear traverse techniques as outlined in ASTM Designation C457-60T. Because the primary concern of this study was the distribution of paste and air contents within the core, each was sectioned as shown in Figure 16 to obtain surfaces at nominal levels of 1/4", 3/4", 1", and 3" below the deck wearing surfaces. These surfaces were designated as A_1, A_2, B_1, B_2, C_1, C_2, and D_1, D_2. The two faces at each level were mirror images separated only by the width of a saw cut plus minor polishing abrasion. This method provided approximately 50 inches of traverse for each face, the combined area of which was approximately 25 square inches. For concrete containing 1" maximum size aggregate, a minimum of 95 inches of traverse is required to obtain a valid estimate of the air void characteristics, based upon statistical considerations. Thus, faces A_1 and A_2, B_1 and B_2, etc. were combined and considered as a single face designated A, B, C, or D, which are considered to be respectively 1/4", 3/4", 1", and 3" below the original surface. Since the concrete near the surface was of special concern, an attempt was made to determine the air void characteristics as close to the surface as possible. Two methods of sectioning were used. For 8 of the 26 cores, slices were cut from the disc bounded by the original surface and face A, perpendicular to these faces. These segments were then embedded in epoxy resin, polished, and a single traverse run on each segment as close to the top as possible. Segments embedded in the resin can be seen in Figure 17, which accompanies a subsequent description. These were designated "surface" properties.

During the course of this work, ASTM proposed a method for determining the properties of the near surface of cores in which the surface is polished away and the measurements made on a face parallel to A_2, B_2, etc. at a level at the base of the texturing asperities. The remaining surface measurements (18 of 26) were made in this manner.

Before the results are discussed, a brief summary of the voids occurring in hardened concrete is necessary. Voids in hardened concrete result from (1) air filled voids in the fresh concrete, and (2) remnants of water filled voids in the fresh concrete. It is not always possible to distinguish between the two, particularly for large voids caused by incomplete consolidation. The voids can usually be separated on the basis of their size and/or shape. It is common to designate three types of voids as follows:

(a) Large voids -- having irregular shape and attributed to incomplete consolidation. These voids may have been water filled in the fresh concrete.

(b) Entrained air voids -- having essentially a spherical shape and represented in the polished surfaces as circles having a diameter of 1 mm or less.

(c) Entrapped air voids -- having an irregular shape and represented in the polished surfaces by spheres having a diameter greater than 1 mm.

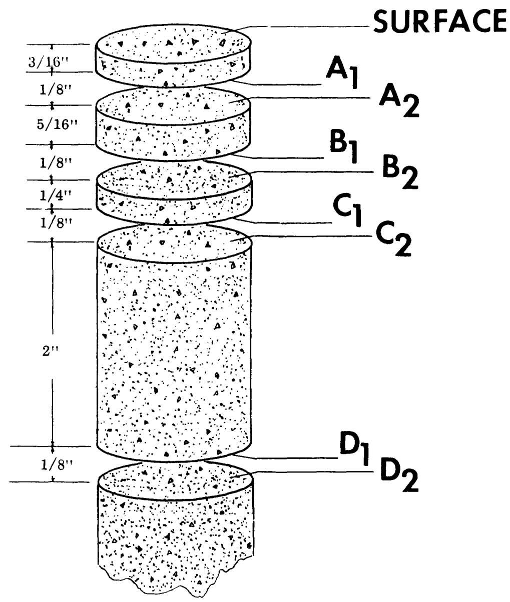

Figure 16. Method of slicing cores and designation of surfaces.

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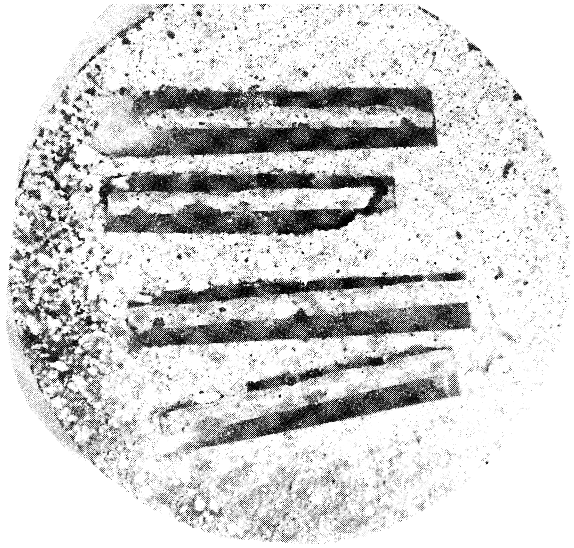
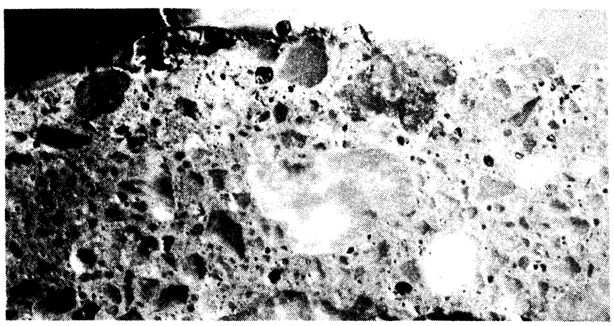
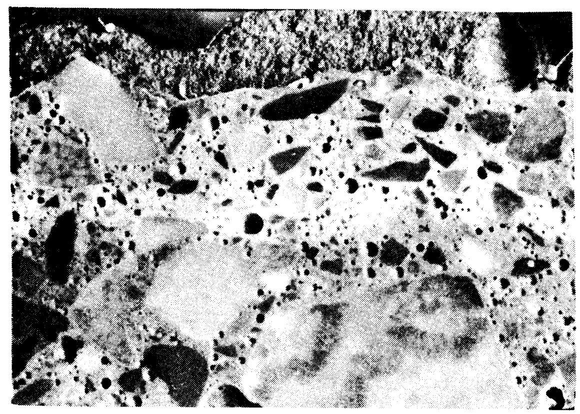


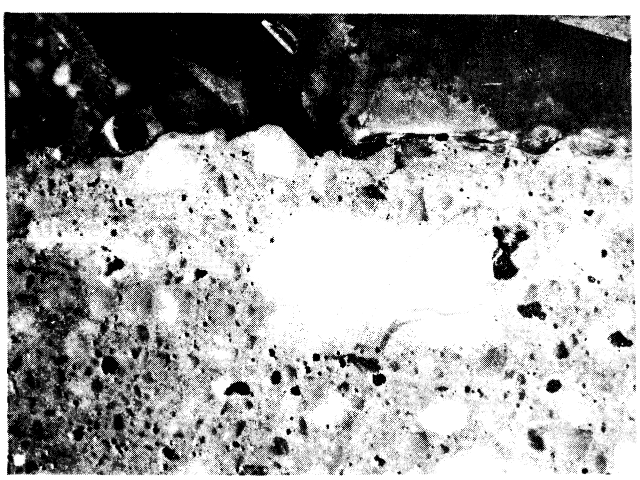
(a) Vertical slices of near surface imbedded in mortar and epoxy.



(b) Close-up of surface.



(c) Close-up of surface.



(d) Close-up of surface.

(e) Close-up of surface.

Figure 17.

This classification is arbitrary and the original condition of particular voids (i.e., water filled or air filled) may not always be clear. In the cores studied, large voids were rare. Thus, in this report voids will be classified as "entrained" or "entrapped", the latter designation including the large voids as described above.

A comparison of the air contents of the hardened concrete and those determined from the corresponding fresh samples is given in Figure 18. The air content shown for each sample of hardened concrete is the cumulative void total for all of the faces of a given core. Thus, it is based upon approximately 400 inches of traverse. Twenty-two of the 26 samples are within 1.25 percentage points of the air content measured in the fresh concrete.

Published research (Powers 1954, Mielenz 1964) indicates that air entrainment may be considered adequate when (1) the volume of air in the mortar fraction, A, is about 9 percent; (2) the number of voids per linear inch of traverse, n, is significantly greater than the numerical value of the percentage of air in the concrete; (3) the specific surface, α, of the voids is greater than 500 in.^{2}/in.^{3}; and (4) the calculated spacing factor, $\bar{L}$, is less than about 0.01 inch.

The data for each of these parameters of the cores are given in Table V. Assuming that a minimum of 8.0 percent air, a number of voids 75 percent greater than the numerical air content, specific surfaces of 500, and a $\bar{L}$ spacing factor less than .0075 in. can be tolerated (these are very tolerant assumptions), it will be noted that only 5 of the 26 cores meet all of the requirements. The principal difficulty was that the amount of air in the fresh concrete was low.

In addition to the low volume of air, the other void characteristics indicate the presence of a relatively high proportion of large voids. These large voids are either remnants from water or what is usually called entrapped air. The dividing line between entrained air, which is beneficial, and entrapped air, which is not, is somewhat arbitrary. If only voids with diameters less than 0.05" (1mm) are considered effective against freezing, then nonbeneficial air ranges from 10 to 50 percent of the total volume measured in the tests reported here.

From the standpoint of durability, perhaps the most significant air void parameter is the distance between voids, since this relates to the distance which water must travel during freezing to relieve the hydraulic pressures generated. Powers (1954) proposed the spacing factor, $\bar{L}$, as a means of estimating the average spacing of bubbles. Calculation of this factor takes into account the amount of air, (A), indirectly the size of bubbles, (α), and the volume of the paste to be protected.

Powers (1954 and 1965) has suggested that the spacing factor must not exceed 0.01" to ensure frost resistance. Mielenz (1964) indicates that for adequate protection the spacing factor should not exceed 0.0055" to 0.0100", the precise value required being dependent upon the characteristics of the concrete and its environment.

The spacing factor cannot be measured in the fresh concrete although a recently reported device has been used experimentally (Torrans and Ivey 1969).

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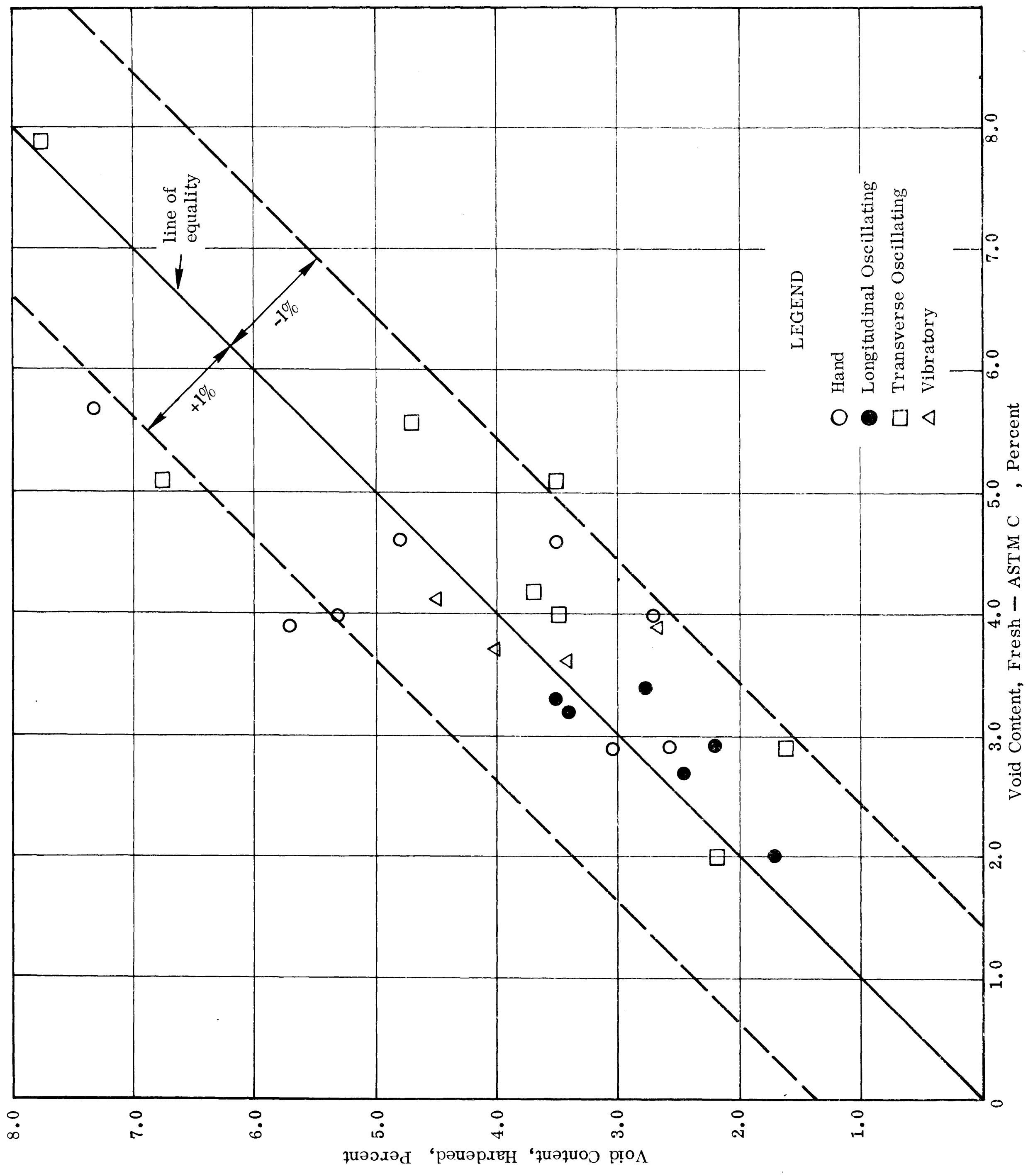

Figure 18. Relationship between the void contents measured in the hardened and freshly mixed concrete.

AIR VOID CHARACTERISTICS OF HARDENED CONCRETE
(Blocked in values meet or exceed given criteria for protected concrete; asterisked samples meet all four criteria.)

Sample	Screeding Method	Air Volume, %				Void Characteristics		
		Fresh	Hardened			Voids / inch of concrete, n	Specific surface, α in.^{2}/in.^{3}	Spacing factor $\bar{L}$, in.
			Total	Mortar Fraction Total	Percent of Total Voids > 1 mm			
1-1	Mechanical vibrating (surface)	3.7	4.02	$\boxed{8.09}$	28.4	3.05	304	0.0162
1-2		3.9	2.66	5.18	32.7	4.33	$\boxed{651}$	0.0089
12-1		3.6	3.37	6.49	36.2	$\boxed{6.70}$	$\boxed{796}$	$\boxed{0.0065}$
12-2*	"	4.1	4.49	$\boxed{8.80}$	33.7	$\boxed{8.00}$	$\boxed{713}$	$\boxed{0.0068}$
2-1*	Mechanical oscillating (transverse)	7.9	7.75	$\boxed{13.91}$	40.9	$\boxed{12.16}$	$\boxed{628}$	$\boxed{0.0038}$
2-2*		5.1	6.76	$\boxed{12.22}$	31.9	$\boxed{11.47}$	$\boxed{678}$	$\boxed{0.0045}$
5-1		2.0	2.21	4.51	11.8	$\boxed{6.59}$	$\boxed{1,012}$	$\boxed{0.0066}$
5-2	"	2.9	1.56	3.13	29.5	$\boxed{2.81}$	$\boxed{722}$	0.0102
15-1	"	4.0	3.49	7.06	41.5	$\boxed{6.91}$	$\boxed{791}$	$\boxed{0.0068}$
15-2	"	4.2	3.72	7.32	11.6	$\boxed{7.03}$	$\boxed{757}$	$\boxed{0.0066}$
16-1*	"	5.6	4.71	$\boxed{9.56}$	14.9	$\boxed{11.26}$	$\boxed{957}$	$\boxed{0.0047}$
16-2	"	5.1	3.52	6.93	30.7	$\boxed{7.62}$	$\boxed{867}$	0.0060
3-1	Mechanical oscillating (longitudinal)	2.7	2.42	4.57	51.2	2.83	467	0.0121
3-2		3.2	3.36	6.50	14.9	5.18	$\boxed{616}$	0.0090
4-1		2.0	1.71	3.53	50.3	2.66	$\boxed{620}$	0.0109
4-2	"	2.9	2.17	4.57	39.2	3.13	$\boxed{578}$	0.0118
9-1	"	3.4	2.75	5.08	27.3	4.01	$\boxed{584}$	0.0089
9-2	"	3.3	3.49	7.44	37.6	4.88	$\boxed{560}$	0.0096
7-1*	Hand	4.0	5.32	$\boxed{10.43}$	14.5	9.29	$\boxed{699}$	$\boxed{0.0064}$
7-2	"	4.6	3.51	6.89	29.3	$\boxed{6.41}$	$\boxed{729}$	$\boxed{0.0071}$
8-1	"	2.9	3.01	6.76	28.2	4.80	$\boxed{623}$	0.0096
8-2	"	2.9	2.47	4.36	20.2	4.25	$\boxed{689}$	0.0096
11-1*	"	5.7	7.30	$\boxed{14.30}$	23.8	$\boxed{15.15}$	$\boxed{830}$	$\boxed{0.0038}$
11-2	"	4.0	2.70	5.26	30.4	$\boxed{5.75}$	$\boxed{852}$	$\boxed{0.0071}$
13-1	"	4.6	4.75	$\boxed{9.69}$	37.7	6.56	$\boxed{551}$	0.0077
13-2	"	3.9	5.71	$\boxed{11.17}$	33.8	5.54	388	0.0092
Ratio of "protected" to total.				7/26		6/26	23/26	13/26

The spacing factors determined from the cores are plotted in Figure 19 as a function of the air content determined on the corresponding fresh concrete. It will be seen that a good relationship exists. One point shows an unusually large spacing factor. Closer inspection of the core (sample 1-1) showed that it did contain a very coarse void system, with practically no small voids, although 72 percent of the air volume was present in voids smaller than 0.05".

If concrete having a spacing factor below 0.0055" is accepted as unquestionably resistant, that with spacing factors between 0.0055" and 0.010" as borderline or variable, and that with spacing factors above 0.010" as unprotected, then an air content of approximately 5.0 percent in the fresh concrete was required to ensure durability. Concretes with air contents between 3.0 and 5.0 percent fall within the borderline area, and those below 3.0 percent are unprotected. These field data substantiate the 1966 revision of the Virginia specifications for concrete for bridge decks to require air in the amount of $6\frac{1}{2} + 1\frac{1}{2}$ percent.

In addition to the air void characteristics of the entire volume, the variation of these characteristics at different levels of the concrete is of considerable concern. It is widely believed that the void system near the surface is damaged by working of the concrete during finishing. Support for this view is found in the results of the comprehensive studies of cores removed from bridges during the study of deck performance (PCA – BPR 1970). This report states:

> While some scaling was attributable to batch-to-batch variation in air content, most of the observed scaling and incipient scaling in cores from air-entrained decks were related to nonuniform air void distribution, particularly in thin, irregular zones at the immediate wearing surface.

Attempts to create these thin irregular zones in the laboratory or to change the surface properties of originally satisfactory concrete have been unsuccessful (Malish, et al 1966). There are published data to suggest that it is extremely difficult to "work out" the entrained air, which suggests that the deficient zones may result from differences in the distribution of air voids within the concrete prior to manipulation rather than from manipulation.

The air contents, voids per inch, specific surfaces, and spacing factors for the cores at various levels are shown in Figure 20. It will be noted that with the exception of one sample (2-2) a lower spacing factor, indicating better protection, was obtained at the surface than at lower levels. It will be seen that the specific surface increased considerably near the surface, as did the number of voids in some cases.

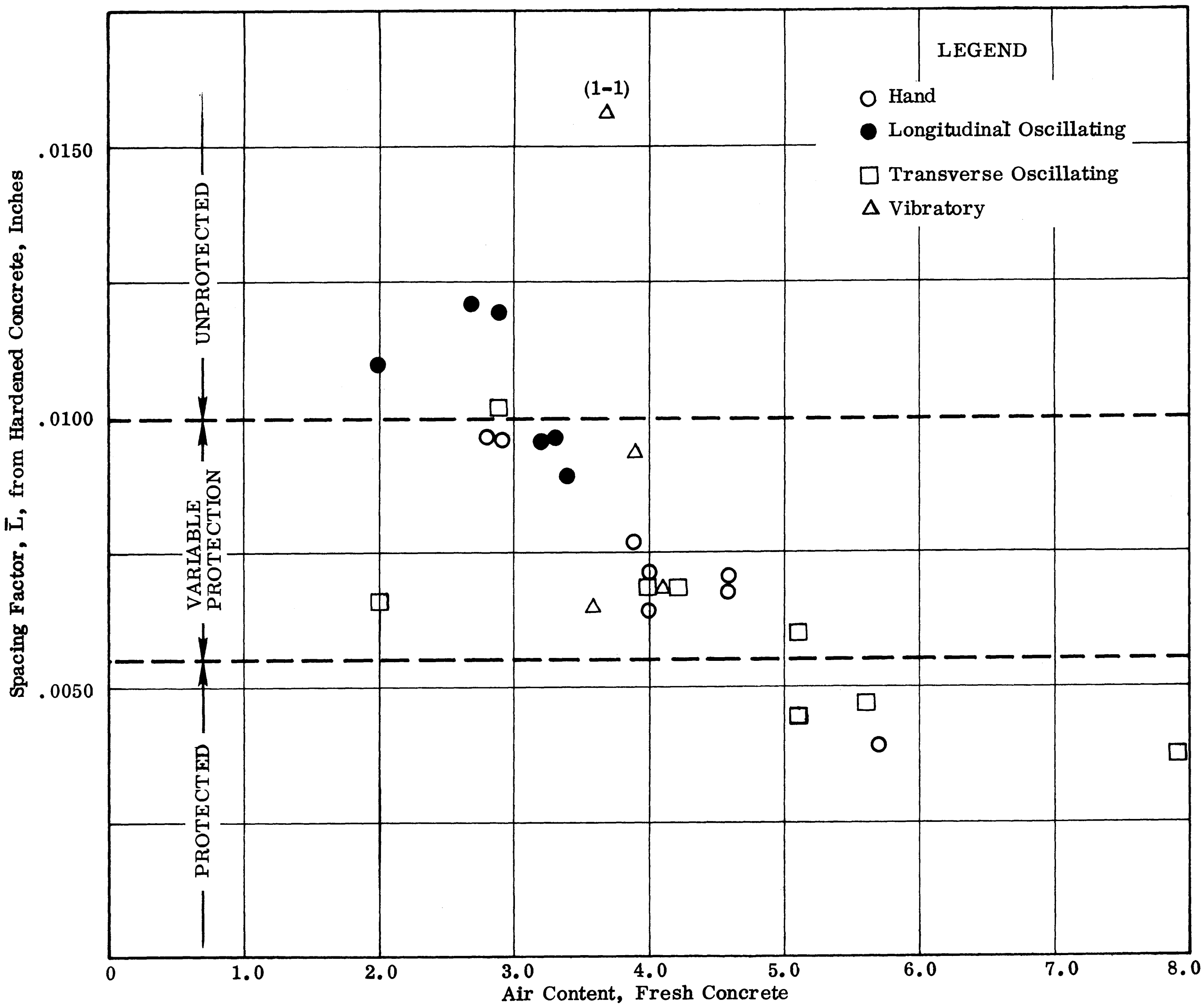

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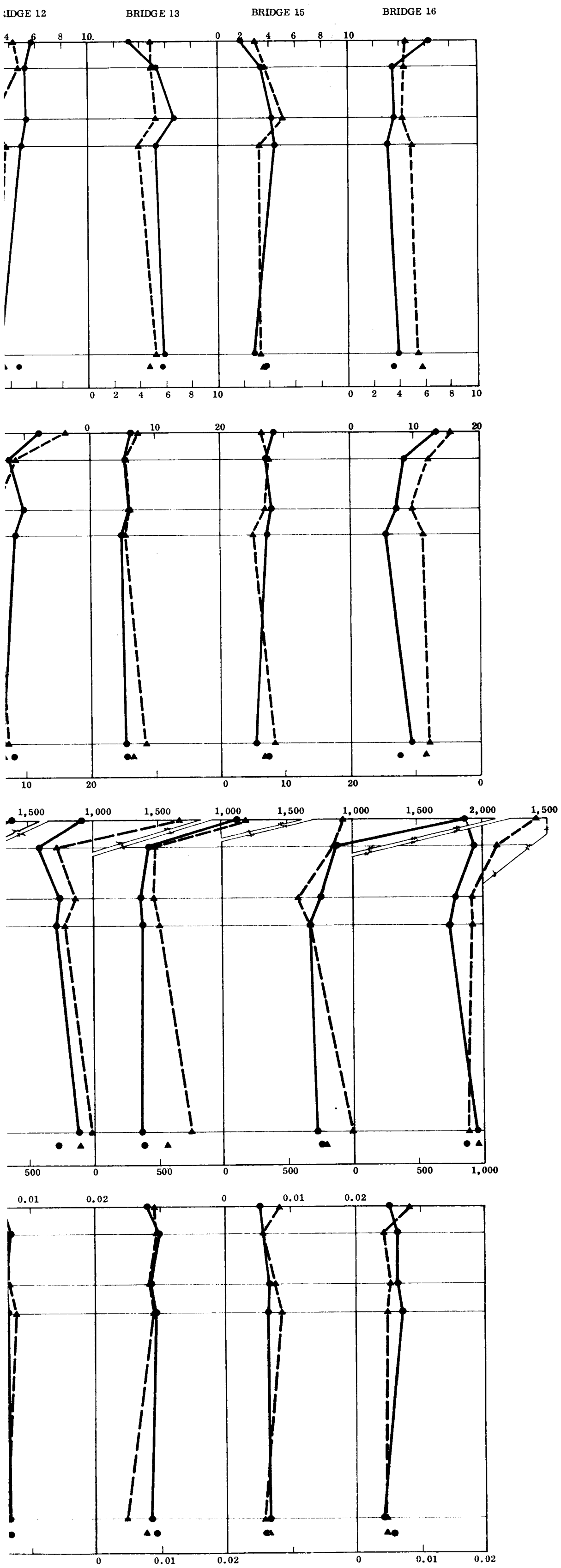
Figure 19. Relationship between the spacing factor measured for the hardened concretes and the air content measured in the freshly mixed concrete.



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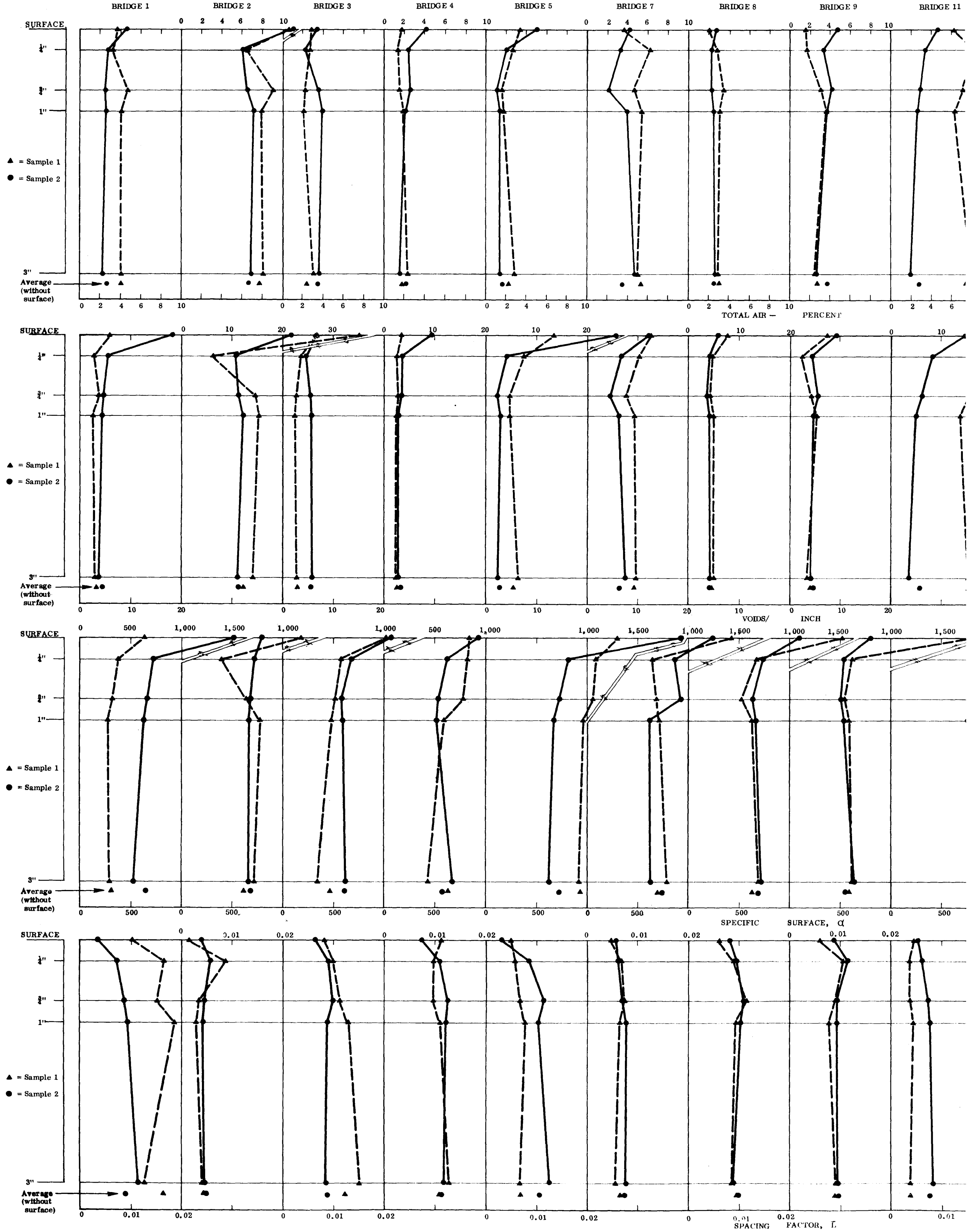

Figure 20. Variation with depth of air void para[meters] measured on the hardened concrete.

The data in Figure 20 indicate that the important air void characteristics within the mass of the concretes remain reasonably constant with varying depths from the surface. At the near surface the volumes of air and specific surfaces increase and the spacing factor usually is smaller. The values in Figure 20 are based upon the total volume of concrete. At the surface, the volume of paste, which contains all of the air, is much higher than in the concrete as a whole, so that more air would be required to provide the same degree of protection. In Table VI the air void characteristics of the surfaces and mass of the cores are presented in terms of air in the cement paste rather than as a percentage of the entire concrete. Assuming that the paste content of the mortar is approximately 50 percent, the criteria for protected concrete would be the same as those given earlier, except that the paste should contain 18 percent air as compared with 9 percent based upon the mortar fraction.

Several points from Table VI are of interest. At the near surface, the air-paste ratios are consistently low and meet the criteria in only 2 of 26 cases. The average void contents were reduced by about 70 percent as compared with the void contents within the mass of concrete. On the other hand, the remaining criteria are met more often in the surface concrete than in the remainder. Most authorities consider parameters other than void content,especially spacing factor and specific surface, to be most important for aassessing the degree of protection of the concrete. Thus, the changes in the air void system indicate that the volume of air is reduced, but that the air lost is in the form of the larger bubbles, which offer little protection. The more important void properties, specific surface, and spacing factor are improved. This finding is consistent with the statement by Mielenz (1964).

> . . . air content in the uppermost part of a concrete slab, particularly within 1/2 inch of the finished surface, is characteristically less than that in the concrete at greater depth because the manipulation removes a large portion of the larger voids. This action may decrease the air void content of the top most 1/16 inch to one-third or less of the air content of the concrete as a whole. Nevertheless, if the concrete was originally adequately air-entrained, the spacing factor is not significantly modified. On the other hand, the specific surface of the void system is increased substantially because of the elimination of the large voids.

To say that the air void properties are better at the surface than at depth is inconsistent with the generally held belief that the air voids in the near surface can be detrimentally affected by surface manipulation; however, no evidence for detrimental changes was found. The findings from PCA-BPR studies suggest that concrete near the surface is more prone to very small areas deficient in air voids or high water-cement ratios. The cores were studied and some uneven distributions of air voids were noted. These were as common in the mass of concrete as in the near surface, which suggests that the unevenness is a function of the mixing process rather than subsequent manipulation.

TABLE VI

AIR VOID PROPERTIES OF CORES AND SURFACES
(Blocked in values meet or exceed given criteria for protected concrete.)

Sample	Concrete (without surface)					Surface				
	P/A	Voids/in. Paste	α	$\bar{L}$	Voids >1 mm.	P/A	Voids/in Paste	α	$\bar{L}$	Voids >1 mm.
1-1	17.55	13.30	304	.0162	8.1	8.67	13.97	645	.0105	0
1-2	12.10	19.71	651	.0089	7.2	11.99	46.37	1547	.0038	0
12-1	15.53	30.89	796	.0065	5.1	10.80	42.26	1566	.0039	1.8
12-2	18.32	32.67	713	.0068	6.9	11.18	29.17	1044	.0058	0
2-1	39.66	62.24	628	.0038	7.7	38.69	134.0	1386	.0019	1.5
2-2	32.54	55.21	678	.0045	5.8	27.02	53.2	787	.0047	0
5-1	8.78	23.80	1012	.0066	1.9	5.66	23.0	1623	.0050	0.8
5-2	7.05	12.70	722	.0102	4.1	10.57	51.5	1945	.0032	0.2
15-1	14.42	28.53	791	.0068	4.1	6.29	14.6	927	.0084	0
15-2	17.00	32.15	757	.0066	2.4	3.67	17.1	1864	.0052	4.9
16-1 (1)	21.53	50.24	957	.0047	2.6	10.21	36.6	1434	.0041	1.8
16-2	15.54	33.65	867	.0060	5.3					
3-1	12.95	15.10	467	.0121	13.5	5.69	20.1	1141	.0065	8.0
3-2	13.55	20.90	611	.0090	3.6	7.05	14.4	1012	.0080	0
4-1	8.62	13.41	620	.0109	9.3	4.06	8.56	843	.0111	18.2
4-2	8.54	12.33	578	.0118	7.5	7.56	17.69	936	.0077	0
9-1	15.40	22.50	584	.0089	7.2	3.65	16.21	1777	.0055	10.5
9-2	14.39	20.13	560	.0096	7.7	8.19	16.33	797	.0087	0
7-1	21.24	37.09	699	.0064	3.3	8.60	30.53	1420	.0048	7.8
7-2	15.50	28.28	729	.0071	6.5	9.72	30.23	1245	.0052	0.6
8-1	11.47	17.87	623	.0096	6.2	4.39	14.97	1362	.0067	2.5
8-2	9.05	15.58	689	.0096	4.8	6.50	14.33	882	.0087	24.0
11-1(1)	30.98	64.30	830	.0038	6.0	15.01	44.14	1177	.0045	20.0
11-2	11.06	23.54	852	.0071	5.5	9.13	29.04	1272	.0052	0
13-1	23.66	32.65	551	.0077	8.1	13.54	20.31	601	.0092	25.5
13-2	27.80	27.02	388	.0092	14.0	8.38	17.22	822	.0083	19.4
Ratio of protected to total	8/26	15/26	23/26	13/26	13/26	2/26	23/26	26/26	16/26	

During the microscopic examination of the vertical slices of the near surface concretes, carbonation was noted in varying amounts among the 13 cores thus sectioned. Typical appearances of three cores can be seen in Figure 17. Although no particular significance is attached to this carbonation, except as a potential contributor to surface shrinkage (Verbeck 1958), the depths were recorded for possible correlations with performance or construction activity. In Table VII the approximate depths of carbonation are shown along with the approximate depths for which the cores were essentially all mortar. As seen in Table VII, the carbonation was confined to this mortar phase except in two cases (cores 5-2 and 9-1). In the case of core 5-2, which will be discussed later, the surface was removed by a hard rain after curing, and was restored with a dry cement shake. No correlations were found between the depths of carbonation and recorded construction operation, although the carbonation seems to be more prevalent when records indicate a contamination of the surface during finishing and "smoothing up" by mortar or paste dragged over the cored area.

Mielenz (1964) states that "intense carbonation extends inwardly from the surface in a superficial zone that is usually less than 1/3 inch (3 mm) thick but this zone is dense, continuous with the cement paste and mortar matrix at greater depth. Such carbonation results from slow interaction of the hardened cement paste with carbon dioxide of the atmosphere". The only contrary indication from these results is that the intense carbonation occurred comparatively early in the life of the concrete.

TABLE VII

DEPTHS OF MORTAR AND CARBONATION OBSERVED IN CORES SECTIONED VERTICALLY

Core	Depth of Mortar (from surface), mm	Depth of Carbonation (from surface), mm	Relationship of Carbonation to Mortar Depth
1-1	0	0	—
12-1	2-3	2-3	equal
2-1	3	0-3	equal or less
2-2	3-4	.5-2	half
5-2	1-3	2-6	twice
15-1	1	1	equal
16-2	0-1	0-1	equal
3-1	2	0-2	equal or less
4-2	1	0-1	equal or less
9-1	0-1.5	1-4	twice
9-2	3-4	3-4	equal
11-2	1-2	1-2	equal
13-2	0-1	0-1	equal

Performance

General

Included in the original sampling were 17 bridges comprising 78 spans. On each bridge detailed data were gathered during construction on only one span, under the assumption that one span would be sufficiently representative of the other spans in the structure. Obviously the construction and weather conditions would not be the same for all spans of a given structure. The number of structures involved, however, would tend to mitigate special influences on a given span. Thus on a given bridge, the studied span might be better, worse, or representative of the remaining spans. Initially, the performance of all spans will be described and then consideration will be given to the behavior of the special study spans. The performance of the individual study spans will then be related to the data gathered during construction of that particular span.

Two of the 17 structures were much larger than the others, containing 8 and 13 spans respectively. The performance of these structures would exert a disproportional influence on the results. The remaining 15 bridges, comprising 60 spans, were more representative of the "typical" bridge used in Virginia and provide a more diverse sampling of performance.

Periodic inspections were made of the decks at intervals of approximately two years. The final observations on which most emphasis is placed here were made during the summer of 1970 when the spans were seven years old, although not all had been under traffic for that period. In fact, one half of a test span has not yet been opened to traffic although the adjacent one-half span has.

As described in an earlier report (Davis, North and Newlon 1971) the seventeen structures were surveyed in 1970 using the procedures developed in connection with the PCA - BPR nationwide survey of decks (BPR--PCA 1969). A discussion and illustration of defects commonly encountered in bridge decks is included as Appendix C. The definitions and methods used in the survey are described in Appendix D. Several techniques were used in the various surveys but the format used in the PCA--BPR study is widely used and permits a readily understandable presentation.

The overall results are summarized in Table VIII. A comparison is made between the defects noted on the 78 spans of the 17 bridges and those observed on 248 spans on 80 bridges from the PCA--BPR sample exposed for approximately the same length of time.

As seen in Table VIII, the two groups of spans have exhibited very similar performance. Thus, the 17 structures included in this study appear to adequately represent the behavior of a large portion of bridges built under similar specifications and procedures. Two of the structures comprising 25 percent of the spans have been resurfaced for reasons that will be subsequently discussed. The fact that one of these contained 13 spans disproportionately influences the seriousness of the need for overlaying the comparatively new decks.

As seen in the data in Table VIII, scaling and cracking are prevalent on the decks. The remaining defects are much less frequent. As will be seen later, the severity of the defects is greater than "light" in nine spans for scaling and in 6 spans for transverse cracking. Surface spalling occurs on two structures but the spalls are large on only one. The occurrences of the various defects on the special study spans are shown in Table IX. The performance of these spans is seen to be typical of the total sample in that scaling and transverse cracking are the most prevalent defects. Data from the 1966 survey are presented in Table X in a similar format. At the time of the 1966 survey, most of the spans had just begun to carry traffic.

TABLE VIII

PERFORMANCE OF BRIDGES IN RANDOM SURVEY SAMPLE
AND
BRIDGE FINISHING SAMPLE

Defect	Occurrence of Defect by Spans		
	PCA-BPR Sample 1961-1970, % of Spans Affected	Finishing Study Sample 1963-1970, % of Spans Affected	Number of Structures Affected in Finishing Study Sample
Covered	17	23	2
Uncovered	83	77	15
No Scaling	5	18	3
Scaling	95	82	12
No Cracking	25	10	1
Cracking	75	90	14
Transverse	59	63	12
Longitudinal	15	30	7
Diagonal	4	8	3
Pattern	23	38	8
"D"	0	0	0
Random	51	55	13
No Rusting	100	100	15
Rusting	0	0	0
No Surface Spalling	90	91	13
Surface Spalling	10	9	2
No Joint Spalling	97	71	14
Joint Spalling	3	29	1
No Popouts	82	82	11
Popouts	18	18	4

TABLE IX

DISTRIBUTION OF DEFECTS ON STUDY SPANS IN 1970

Bridge	Scaling	Cracking					Rusting	Surface Spalling	Joint Spalling	Popouts
		Trans.	Long.	Diagonal	Pattern	Random				
1	30%L	L	M	O	O	O	O	O	O	O
2	O	L	O	O	O	O	O	O	O	O
3	40%L	O	O	O	O	O	O	O	O	O
4	15%L	L	L	O	L	L	O	10S	O	O
5	15%L+	O	L	O	L	O	O	O	O	O
6	35%L*	M	O	O	O	O	O	O	O	f
7	30%L	M	O	O	L	O	O	O	O	O
8	15%L	O	O	O	O	L	O	O	O	O
9	Covered									
10	50%L+	O	O	O	O	O	O	O	O	O
11	50%L**	O	O	O	L	L	O	O	O	O
12	20%L	O	O	O	L	L	O	O	O	f
13	15%L+	L	L	O	O	L	O	O	O	f
14	Covered									
15	20%L	L	O	O	O	L	O	O	O	O
16	0	L	L	O	O	O	O	O	O	O
17	0	O	O	O	O	L	O	O	O	O

*Some heavy scaling on span
**Some medium scaling on span
+Predominantly abrasion

L = Light
M = Medium
S = Small
f = Few

TABLE X

DISTRIBUTION OF DEFECTS ON STUDY SPANS IN 1966

Bridge	Scaling	Cracking				Surface Spall	Popouts
		Trans.	Long.	Pattern	Random		
1	5L	O	L	O	L	O	O
2	O	O	O	O	O	O	O
3	O	O	O	O	O	O	O
4	5L	O	O	L	O	O	O
5	10L	O	O	L	O	O	O
6	20L	O	O	O	O	O	O
7	O	L	O	O	O	O	O
8	O	O	O	O	O	O	O
9	10L	L	O	L	O	O	O
10	20L	O	O	O	O	O	O
11	O	O	O	L	O	O	O
12	O	O	O	O	O	O	f
13	O	O	O	O	L	O	f
14	Covered						
15	O	L	O	O	O	O	O
16	O	O	O	O	O	O	O
17	O	O	O	O	O	O	O

The occurrences and severity of the various defects on all of the 60 spans as observed in the 1970 survey are shown in Tables XI -- XIII. The specific defects will subsequently be discussed in detail, but from the data in the tables it can be seen that, with only a few exceptions, when a defect occurs on one span of a structure it usually occurs to about the same extent on all spans of the structure. Surface spalling, for example, occurs on only two bridges, but it occurs on all spans of bridge 4.

The defects will now be considered individually and the interrelationship among the performance, materials, and construction discussed.

TABLE XI

DISTRIBUTION OF SCALING AND SPALLING ON ALL SPANS
OF THE STUDY BRIDGES — 1970
(Study spans are indicated by boxes.)

Bridge	Span 1	2	3	4	5	6	7	8
1	40	$\boxed{30}$	40	95	—	—	—	—
2	O	O	$\boxed{O}$	—	—	—	—	—
3	5	5	10	80(1)	$\boxed{40}$	—	—	—
4	$\boxed{15}$	15(1)	O	O	—	—	—	—
5	20*	$\boxed{15*}$	15*	15*	—	—	—	—
6	20	20	20(2)	$\boxed{35(2)}$	—	—	—	—
7	50	30	30	30	40	30	$\boxed{30}$	30
8	25	$\boxed{15}$	10	20	—	—	—	—
9	Covered							
10	50*	50*	$\boxed{50*}$	—	—	—	—	—
11	70(2)	$\boxed{50(1)}$	50(1)	60(1)	50(1)	—	—	—
12	$\boxed{20}$	20	35	—	—	—	—	—
13	15	5	15	$\boxed{15}$	—	—	—	—
14	Covered							
15	20	$\boxed{20}$	20	—	—	—	—	—
16	0	$\boxed{0}$	0	—	—	—	—	—
17	0	$\boxed{0}$	0	—	—	—	—	—

* Predominantly abrasion rather than scaling

(1) Some medium scaling

(2) Some heavy scaling

Surface Spalling

Bridge	1	2	3	4	5	6	7	8
4	$\boxed{10S\ 0L}$	5S 4L	5S 5L	6S 2L	—	—	—	—
5	1S 0L	$\boxed{0}$	0	0	—	—	—	—

Joint Spalling

Bridge	1	2	3	4	5	6	7	8
3	1L	1S 1L	1S	2S 1L	$\boxed{O}$	—	—	—

TABLE XII

DISTRIBUTION OF TRANSVERSE (T) AND LONGITUDINAL (L) CRACKING ON ALL SPANS – 1970
(Study spans are indicated by boxes.)

Bridge	Span 1 T	Span 1 L	Span 2 T	Span 2 L	Span 3 T	Span 3 L	Span 4 T	Span 4 L	Span 5 T	Span 5 L	Span 6 T	Span 6 L	Span 7 T	Span 7 L	Span 8 T	Span 8 L
1	L	L	L	M	M	O	L	O	–	–	–	–	–	–	–	–
2	L	O	L	O	L	O	–	–	–	–	–	–	–	–	–	–
3	L	O	L	O	L	O	L	O	O	O	–	–	–	–	–	–
4	L	L	L	L	L	L	L	L	–	–	–	–	–	–	–	–
5	O	O	O	L	L	L	O	O	–	–	–	–	–	–	–	–
6	O	O	L	O	L	O	M	O	–	–	–	–	–	–	–	–
7	L	L	L	O	L	O	L	O	M	L	M	L	M	O	M	O
8	O	O	O	O	O	O	O	O	–	–	–	–	–	–	–	–
9	Covered															
10	O	O	O	O	O	O	–	–	–	–	–	–	–	–	–	–
11	O	O	O	O	L	O	L	O	O	O	–	–	–	–	–	–
12	O	O	L	L	L	O	–	–	–	–	–	–	–	–	–	–
13	L	L	L	O	O	L	L	L	–	–	–	–	–	–	–	–
14	Covered															
15	L	O	L	O	L	O	–	–	–	–	–	–	–	–	–	–
16	O	L	L	L	O	L	–	–	–	–	–	–	–	–	–	–
17	O	O	O	O	O	O	–	–	–	–	–	–	–	–	–	–

L = Light

M = Medium

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TABLE XIII

DISTRIBUTION OF RANDOM (R) AND PATTERN (P) CRACKING IN ALL SPANS — 1970
(Study spans are indicated by boxes.)

Bridge	Span 1		2		3		4		5		6		7		8	
	R	P	R	P	R	P	R	P	R	P	R	P	R	P	R	P
1	O	L	O	O	L	O	L	O	—	—	—	—	—	—	—	—
2	L	O	L	O	O	O	—	—	—	—	—	—	—	—	—	—
3	L	O	L	L	L	L	O	O	O	O	—	—	—	—	—	—
4	L	L	O	L	O	L	O	L	—	—	—	—	—	—	—	—
5	L	L	O	L	O	L	O	O	—	—	—	—	—	—	—	—
6	O	O	H	O	L	O	O	O	—	—	—	—	—	—	—	—
7	L	L	L	L	O	L	O	L	O	L	O	O	O	L	O	M
8	L	O	L	O	L	L	L	O	—	—	—	—	—	—	—	—
9	Covered															
10	O	O	O	O	O	O	—	—	—	—	—	—	—	—	—	—
11	L	L	L	L	L	O	L	O	L	O	—	—	—	—	—	—
12	L	L	L	L	L	L	—	—	—	—	—	—	—	—	—	—
13	L	O	L	O	L	O	L	O	—	—	—	—	—	—	—	—
14	Covered															
15	L	O	L	O	L	O	—	—	—	—	—	—	—	—	—	—
16	O	O	O	O	O	O	—	—	—	—	—	—	—	—	—	—
17	L	O	O		O		—	—	—	—	—	—	—	—	—	—

L Light

M Medium

H Heavy

Resurfacing

Two of the study structures have been resurfaced. The reason for both of these overlays was failure to obtain sufficient cover over the uppermost reinforcement. In one case (bridge #14), the overlay was placed prior to opening to traffic. In the other case (bridge #9), the spans had carried traffic for four years, at which time extensive repairs to the concrete were made and the decks overlaid. In both cases, the decks were screeded with a full length mechanical oscillating screed. In fact, the photograph in Figure A-2 shows the screeding of the span of bridge 14, in which the steel was subsequently found to have no cover. (See Figure 21.) Subsequent studies reported that the steel elevations were in error and that the deck thickness was shy by 2". Resurfacing prior to traffic was required. It seems incredible that the lack of cover was not noticed during the intensive observations and testing of research, field and contractor personnel on bridge #14; however, it was not. Of course, if steel elevations were actually in error, the type of screed would not have mattered. The similarity of behavior of three of the four bridges screeded with the full length screed suggests at least some association exists.

As a result of similar experiences on other jobs, it is now required practice to move the screed completely over the deck just prior to placement, at which time the distance to the upper steel is checked. Probes of depth are now also made. In spite of these precautions, situations have been encountered in which the upper steel does not have the intended cover. An extensive research project at the Council has been directed toward solving this problem.*

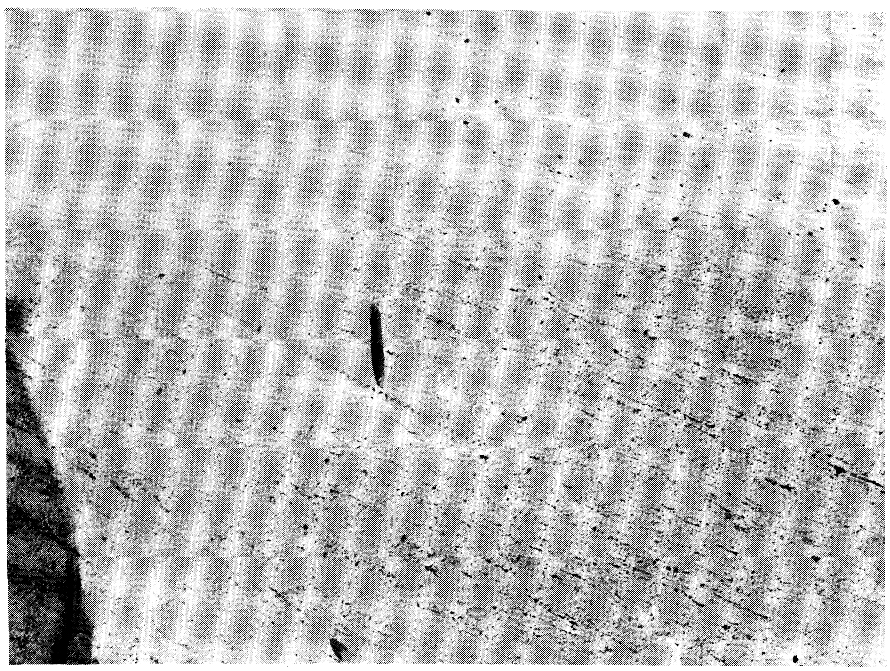

Figure 21. Reinforcement without cover, prior to traffic on bridge #14.

*The portion of this report which discusses the problems related to the use of longitudinal screeding equipment was prepared by M. H. Hilton, highway research engineer. A more detailed discussion will be found in Hilton's report, June 1971.

Screeding equipment which travels transversely is most often used on simple spans of 100' or less, though it has been used on spans of greater length. The transverse screed rails supporting the machine are normally set to the finished grade at each end of the span. The finished elevation of intermediate points on the deck are set on the longitudinal strike off edge of the screeding machine. Assuming structural stability of the machine, these elevations remain fixed and are independent of the girder deflections occurring during concrete placement. Consequently, the thickness of the concrete deck is dependent upon two major factors that should be recognized during construction. These are:

1. The differential temperatures existing between the top and bottom flanges of the girders during concrete placement as opposed to those that may have existed when the forming elevations were established.

2. The transverse position of the concrete dead loading at the time a final screeding pass is made over a given point on the span.

The possible influence of the first factor is illustrated in Figure 22. If no temperature differential exists between the upper and lower flanges of a simply supported bridge girder, it would be in a thermally neutral position (Figure 22a). Due to solar radiation, differential temperatures will generate expansive forces in the upper flange, which are resisted by opposing forces in the lower flange. The resulting effect is an upward deflection of the girder (Figure 22b). If the deck forms were established to grades complying with the neutral position of the girder, but the concrete deck screeded to grade under differential thermal conditions, the thickness of the deck would be decreased by an amount Δ (Figure 22b).

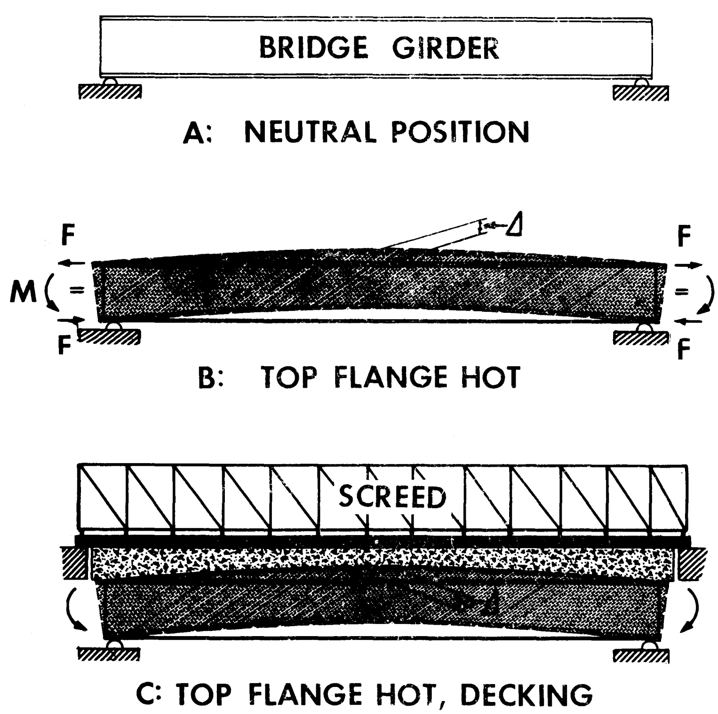

Figure 22. Effects of differential temperatures on a deck screeded with a longitudinal screed.

The influence of the second factor is illustrated in Figure 23. Conventional design procedures for calculating dead load deflections normally assume that each girder is free to deflect independently of other girders in a bridge span. Under partial transverse loading conditions such as the example shown, however, the conventional calculation method yields a midspan transverse deflection pattern markedly different from the actual field deflection pattern. Thus, if the concrete were struck off to grade over the first girder, the midspan deck thickness at this point would be decreased by the difference between the two deflection curves. In addition, the finished grade at this point would be low by an identical amount when all the deck concrete has been placed.

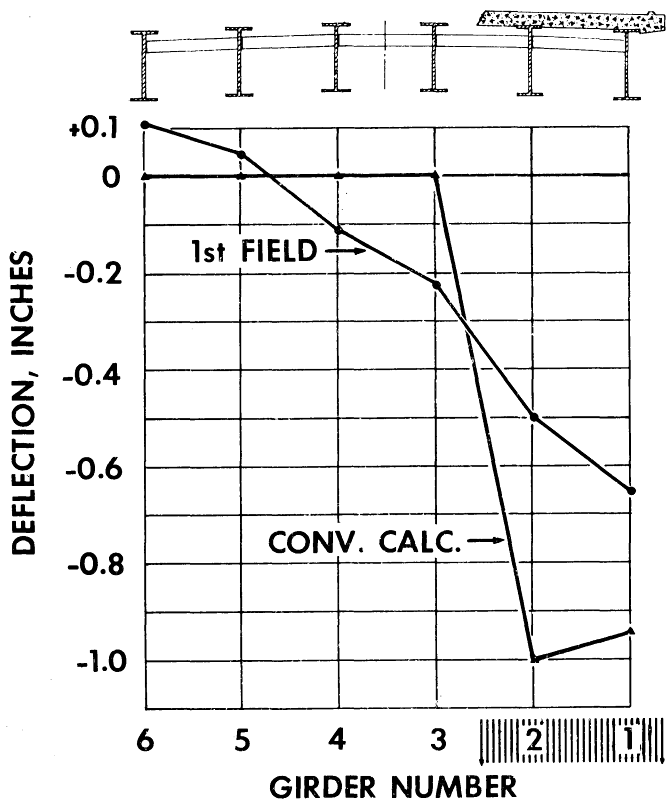

Figure 23. Comparison of conventionally calculated deflections with that measured in the field.

Neither of the two factors discussed above can be exactly compensated for during construction, but their effects can be minimized by observing the following practices. First, forming elevations should be established when the thermal conditions on the girders will approximate those anticipated at the time of concrete placement; and/or the deck forms should be adjusted vertically at a time when the thermal condition of the girders will approximate the condition expected to prevail at the time of concrete placement. The latter precaution is most important since the in-place forming will shield the lower portion of the girders from solar radiation. Differential thermal effects can be virtually negated, of course, by very early or very late concreting operations. Secondly, the final strike off pass of the screeding machine over any given area should lag behind concrete placement by at least three girder spaces -- preferably more. For exceptionally wide roadway widths (more than the equivalent of three 12-foot traffic lanes) the bridge designer should be consulted.

The mechanism responsible for deficient thickness can also be used to explain deficient cover over upper reinforcement. It is significant that the two structures resurfaced (bridges 9 and 14) and the one affected by spalling (bridge 4), which also contains deficient cover, were all screeded with the type equipment illustrated in Figures A-2 and 22. Deficient cover or spalling were not observed on bridges screeded with other types of equipment.

In the case of bridge #9, the lack of cover, and also accompanying shy thickness, was not so general or of sufficient magnitude to affect the structural adequacy of the deck. The structure was accepted with a penalty based upon the deficient thickness and carried traffic for four years prior to resurfacing. During the inspection of 1965 when the bridge had been carrying traffic for approximately one year, rust stains, reflecting the corrosion of the upper steel, were noted at scattered locations, particularly in the northbound lane. There were no evidences of this rusting in the test span. With respect to cover, the test span appeared to be much better than the other spans.

In addition to the lack of cover, numerous repairs were required adjacent to the joints as a result of a joint design detail which was subsequently corrected. It was actually the corrective action necessary at the joints, coupled with the deficient cover, which led to the major repair and resurfacing. No surface spalling was observed because the steel was at the surface rather than at a depth sufficient to promote such spalling.

A view of the repair prior to overlaying in span 10 of the northbound lane is shown in Figure 24. As noted earlier in discussing the variations in the compositions of the mixtures, there is evidence from the measured yields that this concrete contained more water than was called for, and that the deviations from the intended quantities of aggregates and other components were significant. If the concrete in the study span was representative of the concrete in the remaining spans, the deviations cited could have accelerated development of the various problems.

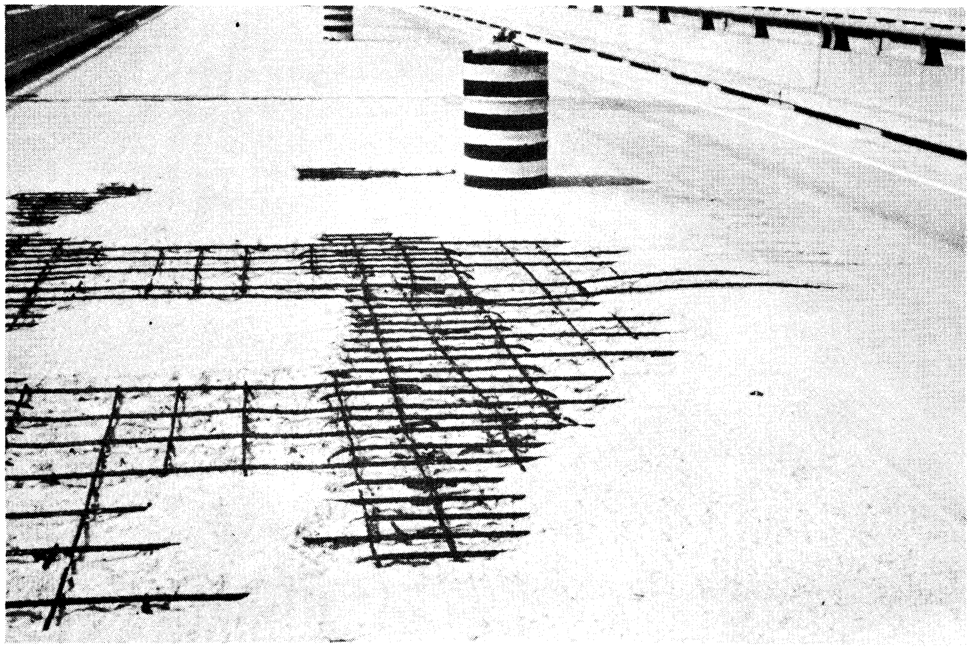

Figure 24. View of repaired areas in span 10 of NBL adjacent to the study span of bridge #9.

Scaling

Only three of the study spans (2, 16, and 17) are free of scaling and these occur on the only three bridges which are completely free of scaling. The prevalence of scaling is largely a result of the generally deficient air void systems discussed earlier. The important role that properly entrained air plays in the prevention of scaling is emphasized by the fact that the three scale-free bridges were the few that, on the basis of the air void system in the hardened concretes, had definitely acceptable spacing factors and other void parameters. The five lowest spacing factors were in order 2-1, 11-1, 2-2, 16-1, and 16-2. No cores were taken from bridge #17 but the measured air contents were 4.8 and 4.9 percent, respectively. Bridge #11 is extensively scaled, with some being of moderate variety. As noted in Table V, the air void system of sample 11-2 is not satisfactory. There is also evidence from the compositional variations discussed earlier that the water-cement ratio was high. This scaling, despite the presence of modest air entrainment, is further understandable because the curing on this span was severely delayed. The two samples shown earlier in Figure 10, for which the delay between texturing and curing exceeded five hours, were from this span. Despite the use of curing compound on this job, curing was delayed until the concrete was sufficiently hard to walk on -- apparently under a misunderstanding of the fundamental purposes of curing. Actually, despite these abuses there is very little scaling in the immediate area of sample 11-1. No core was removed from bridge #6, but the measured air contents were comparatively low at 4.1 and 4.3 percent. It is interesting to note that bridge #6 received a silicone treatment. At the time of construction silicone treatments were being widely promoted for prevention of scaling. Their promise was not realized (NCHRP 1970) and certainly they were not effective on this structure. The most generally or severely scaled structures, in addition to bridges #6 and #11, were bridges #1 and #3, both of which had the highest spacing factors measured. Subsequent to completion of this project, the use of linseed oil treatments for protection of bridge decks became general practice. Some of the spans have received such treatments. Those so treated are indicated in the figures of Appendix B. No significant influence of these treatments on the incidence of scaling is seen in Table XI. The benefits of linseed oil have, however, been shown in previous research (Newlon 1970).

On the basis of the observation to date, good air void characteristics of the total concrete are well correlated with the absence of scaling. Subsequent deterioration of the air void system by manipulation of the surface was not a factor in this project.

Transverse and Longitudinal Cracking

Transverse cracking of "medium" severity exists on study spans of bridges #6 and #7. Additionally, transverse cracking of the "medium" severity level was found on bridge #1, along with "medium" longitudinal cracking on the study span of this bridge. Cracking on other structures, while frequent, is all classified as "light". As shown in Table I all three of the affected structures are on heavily traveled roads (VPD$\approx$ 25,000). Traffic is not believed to be the causative factor, however, because other bridges (#12, #15, #16, #13) with equivalent traffic counts do not show such severe cracking. Reference to Table I will also show that the affected bridges are of different structural types, and that others of similar types are not affected.

Bridges #1, #6, and #7 are all in the same geographical area and were built with concrete from the same materials and from the same plant. Excessive cracking was noted prior to opening to traffic of concrete pavements being constructed concurrently with the bridges studied and using aggregates from the same source. A study of this cracking previously reported (Newlon 1965) indicated thermal or "morning" cracking, and the results from a limited research study which the situation precipitated (Brown 1965) suggested that the clay content of the sand may have aggravated the cracking tendency. Contributions to the shrinkage of concrete by sands with sand equivalent values comparable to those found in the special studies have been reported (Hveem and Tremper 1957). Additional washing of the sand reduced cracking in the pavements constructed subsequent to the placement of the decks on bridges #1, #6, and #7. The presence of medium cracking only on the three bridges made with a sand which had characteristics shown by other studies to influence cracking, strongly suggests a connection between the sand and the cracking.

Longitudinal cracking is slightly more prevalent on the study bridges than was found in the BPR--PCA survey, but all such cracking is light, except that on bridge #1. Actually most of the cracking designated as longitudinal might easily be designated as random but the longitudinal trend most closely describes the appearance. No structural type of construction feature is common to the structures showing longitudinal cracking.

Surface Spalls

Surface spalls or "fracture planes" are currently of considerable nationwide concern. As noted in Table VIII spalling is not extensive in either of the groups of bridges surveyed in detail. There are currently under repair in Virginia several examples of this serious deficiency, however, so its occurrences on bridge #4 warrant discussion.

It is obviously significant that spalling occurs on only one bridge, but it occurs on every span of that bridge. There is nothing unique about the structural aspects of this bridge, and its traffic count is very low ($\approx$ 1600 VPD). It thus appears that some materials or construction detail is responsible for the spalling. About one-half to two-thirds of the spalling on bridge #4 is associated with bent-up short bars near the joints. These are oriented longitudinally, have little or no cover, and reflect an apparent error in locations. The remaining spalls are in areas of the spans away from the joints. In these cases the cover is greater but in general appears to be of the order of 1". Examples of these cases are shown in Appendix Figure C-5.

There is no spalling on the test slab of bridge #5, but there is serious spalling on the parallel structure, which was a part of the same contract. Because the test spans on bridges #4 and #5 were constructed on the same day with material from the same plant it is difficult to separate the discussion of the two structures even though two different screeding methods were used.

For each job, a brief summary report was prepared immediately after construction. It is best to let the reports for bridges #4 and #5 speak for themselves.

> General Comments Concerning 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.

2680 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. 4

(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 south side and was completed at approximately 12:30. 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, which 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 changing to high sky with moderate winds of about 8 miles per hour. The atmospheric conditions contributed slightly to drying.

(3) Concrete Supplied by Local Ready-Mix Plant. 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 given in general comments relating to Jobs 4 and 5, the concrete was rather stiff during the initial part of

the pour. The mix design was based upon a 7% sand moisture. 2681
With the truck following that which delivered sample 4-2, 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 $2\frac{1}{2}$ ounces per yard. With these corrections in the mix it became more workable and the slump changed from about 2 inches to about $3\frac{1}{2}$ inches. Both of our samples came from the first type of concrete. At 10:30 it was discovered that the median which had been poured between 8:00 and 9:30 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 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 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 giving 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 a result of 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.

Memorandum Report Job No. 5

(1) General Concreting Operation. Placement began at the east end of the slab at approximately 8:00 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 pertained 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 which existed 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 of screed poles which were 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 one 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 resulted to a lesser degree in several places and there are on this bridge deck 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.

If spalling or any other defect had developed on the study span of bridge #5, it would have been explainable by one of the many problems that beset its construction. The significant features of bridge #4 were what was termed "excessive deflection" (i.e., greater than expected) along with the comparatively stiff concrete. Assuming that the reinforcing steel was originally at the proper level, the movement of this steel accompanying the significant deflection would be resisted by the relatively stiff concrete so that a weakened plane at the level of the reinforcement might result. This can be visualized in Figure 25. It was apparent that the stiff concrete surface was difficult to close as seen in Figure 26. While the factors influencing the spalling are uncertain, deficient cover is the cause and the observations are consistent with the mechanism described earlier based upon research conducted at the Research Council by Hilton (1971).

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Figure 25. Concreting in progress on bridge #4 using longitudinal screed. The deflection conditions shown earlier in Figure 23 are readily visualized.

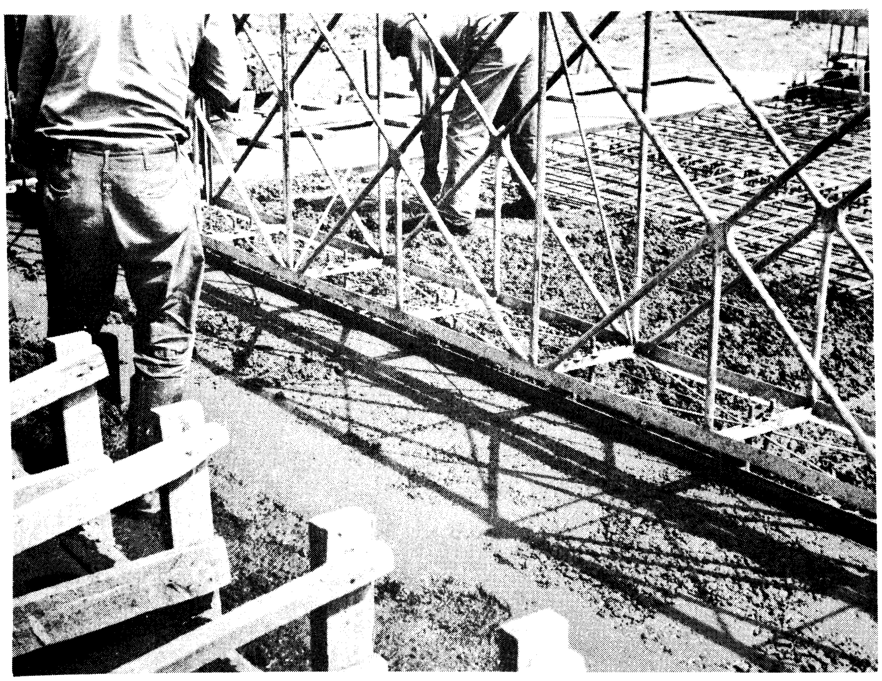

Figure 26. Evidence of stiff concrete on bridge #4.

Random and Pattern Cracking

Random and pattern cracking are likewise prevalent but light. Most of the random cracking appears to be related to plastic shrinkage. Pattern cracking is usually ascribed to improper curing. With the exception of bridge #4, notes made during construction indicate that on each of the spans where pattern cracking is prevalent curing was either late or the coverage was poor. In the case of bridge #4, the curing was applied fairly early and the coverage was adequate, but as noted previously the drying conditions were severe and the concrete very stiff. Minor plastic cracking was noted on the day following construction. The problems surrounding bridges #4 and #5 have already been discussed in some detail.

Although the sample is limited, there is an indication that the occurrence of pattern cracking may be associated with the more heavily carbonated samples listed in Table VII.

Figure 27 shows a view of span 2 of bridge #5 following the rainstorm. Figure 28 shows a view taken in 1971 from approximately the same location. Actually the performance of the span under comparatively heavy traffic has been better than would be expected. As noted in the description of the project, sample 5-2 came from an area in the foreground of Figure 27. A close-up of a vertical section through the surface is shown in Figure 29. The intermingling of the curing compound with the reworked surface is obvious. Also obvious is the network of fine cracks which obviously will manifest at the surface as pattern cracking. The termination of the cracks at the intersection with the curing compound below the surface is striking.

No single cause for pattern or random cracking has been established, but early and complete curing appears to be an important preventive measure. No evidences of unstable aggregates were found in these studies.

Figure 27. View of study span of bridge #5 following the rainstorm — 1963.

Figure 28. View of same area as Figure 27 — 1971.

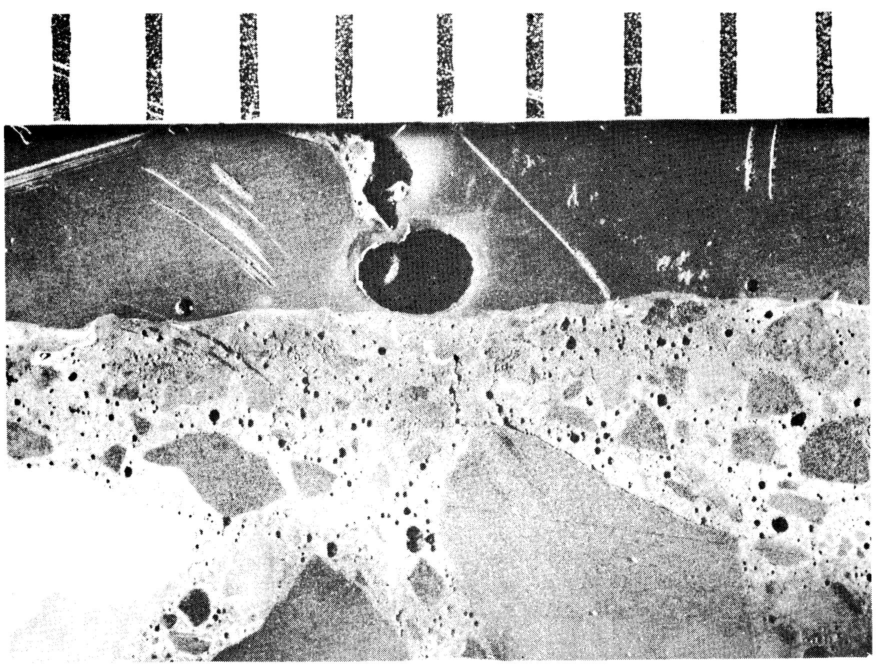

Figure 29. View of near surface of hardened concrete from sample 5-2. Note fine cracking terminating at curing compound,which is intermixed with concrete. Large void at the top of the picture is in the epoxy mounting.

Popouts

Popouts occur on only four structures and are most significant on bridges #12 and #13, in which the coarse aggregate was a chert gravel, a good portion of which was of very low specific gravity. This material and its behavior have been thoroughly discussed in an earlier report (Newlon, Ozol, McGhee 1965).

The altered chert contents of the cores from these two bridges were determined microscopically. The results expressed as volume percentages were as follows:

Sample 12-1 — 14.7%
Sample 12-2 — 18.2%
Sample 13-1 — 22.2%
Sample 13-2 — 15.5%

All of these cores contain sufficient chert to cause popouts. Two incidental observations relating to the formation of popouts on the structures are significant. On bridge #12 a portion of each span has not yet been opened to traffic due to failure to complete the connecting roadway. Although the traffic has removed most of the popped off mortar cones, the untrafficked areas have essentially the same frequencies of popouts or incipient popouts.

The chert contents of the cores from bridge #13 were about as high as are usually found in the aggregates used in the work. Actually there are many fewer popouts on this bridge deck than are usually associated with chert contents of the magnitude shown above. On the same lane as this bridge and approximately 100 yards further on is another structure carrying exactly the same traffic and built under the same contract. The surfaces of this structure contain thousands of popouts, some very large, whereas the four spans of bridge #13 have relatively few. Since it is unlikely that the chert contents of the structures are significantly different, it appears that the difference lies in the fact that the slabs on bridge #13 were constructed during midsummer while those on the other structure were placed in the middle of November and December when freezing occurred early in the life of the concrete at a time when it was highly saturated. It was noted during placement of bridge #13 that the coarse aggregate was being batched in a very dry condition. Undoubtedly, the degree of saturation of the aggregates and the concrete in bridge #13 was much lower than that of the concrete used on the other structure. There is some evidence from other Council studies that such drying is beneficial to the performance of cherts and other such aggregates during freezing and thawing (Newlon, Ozol, McGhee 1965).

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CONCLUSIONS

The results of this study furnish some quantitative date in an area where considerable speculation has existed. While the results are undoubtedly influenced by local conditions, the author is of the opinion that similar studies in other geographical areas would yield equivalent results.

The results are, in general, compatible with and substantiate the findings of other studies in which valid tests of the fresh concrete were usually not available. The principal conclusions are:

1. When viewed against the perfection desired by the engineer, the performance of these decks has been disappointing or borderline. When viewed against the performance that would be expected from concrete with the properties observed, the performance has been better than might be expected.

2. The performance of the 17 structures in this study closely parallels the performance of a large sample of bridges included as part of a nationwide study of bridge deck performance.

3. The primary cause of variable or borderline performance of concrete in bridge decks is variable or borderline fresh concrete. Many of the deficiencies have been overcome by changes in specifications and procedures instituted since the construction of the bridges included in this study.

4. Even with the use of elaborate mechanical equipment, diligent attention must be given to the details of accepted practices of good concreting such as maintenance of low water-cement ratios, adequate air contents, and prompt and thorough curing.

5. The agreement of properties such as unit weight and air content measured in both the freshly mixed and hardened concrete is acceptable for engineering purposes.

6. No influence of the screeding method on the properties of the concrete in place was found. However, several indirect relationships may exist. These include:

   (a) The average slump of concrete placed on jobs using mechanical screeding equipment was 2.8 while that for jobs utilizing hand methods was 3.7. To the extent that slump reflects water content, the use of mechanical screeds should result in a better quality concrete.

   (b) Of the four bridges screeded with the longitudinal screed, three have shown relatively serious deficiencies. Two have been resurfaced primarily because of deficient cover over the uppermost reinforcement. The third span has surface spalling, which also appears to be related to insufficient cover over reinforcement. Hopefully research nearing completion at the Research Council will shed light on this problem and suggest means for eliminating it.

7. Traffic volumes and design features seem to have had little influence on the adverse performance of the seventeen bridges in this study.

8. The only three bridges that are free from scaling were the only three bridges that contained an adequate entrained air voids system.

9. For the class of mixtures used in these decks, a minimum air content of 5 percent was found to be necessary in order to provide a void spacing factor of 0.0055", while air contents of 4 percent provided spacing factors below 0.0075".

10. The importance of the early application of curing was reflected in the scaling of several decks of apparently satisfactory concrete to which curing was applied very late.

11. Uncertainty exists as to the exact proportions of the components, especially water, in the concrete in these bridge decks. Procedures established since this project should improve this situation.

12. The influences of water-reducing admixtures on retarding the setting and reducing the water requirement were apparent in the samples from this project, as was the accelerating effect of high mixture temperature.

13. The data from three spans suggest that the cracking common to them might be explainable from high sand equivalents of the fine aggregates used.

14. Popouts were confined to structures using aggregates previously known to be susceptible to this type of defect.

IMPLEMENTATION

As noted throughout the report, close liaison with operating divisions within the Virginia Department of Highways was maintained throughout this study. Thus many of the findings have already been implemented by changes in specifications or operating procedures. These changes were not made entirely as a result of this research but rather from the mutual efforts represented in this research, experience within the various operating divisions, and results from the nationwide emphases on various bridge deck problems.

Implementation was accelerated by the close relationship which exists between the Council and the Virginia Department of Highways, especially as it is manifested through the various Research Advisory Committees. In addition to these groups, implementation was materially aided by the presentation of the preliminary findings from the field measurements on fresh concrete in instructional sessions held in each construction district in 1964 and attended by about 300 operations personnel.

Among the changes accompanying or influenced by this research are:

1. Upgrading of the requirements for bridge deck concrete as reflected in Table II of this report. The most significant changes were made in 1966.

2. Emphasis on a certification program in portland cement concrete for Departmental and contractor personnel. This program was begun in the fall of 1962, just prior to the work reported here, and has significantly improved all aspects of concrete production.

3. Focusing attention on the critical nature of bridge deck concrete through various meetings, short courses, and publications within the Department.

4. Requiring mechanical screeding of bridge decks.

5. Using monomolecular film to encourage early curing.

RECOMMENDATIONS FOR FURTHER RESEARCH

Most of the major difficulties, specifically lack of cover and variable quality, would be mitigated by the use of bonded two course construction as described in the PCA — BPR Report #6 (1970) and NCHRP Synthesis #4 (1970). Field evaluations of this type of construction are recommended.

ACKNOWLEDGEMENTS

A field project of this magnitude requires extensive coordination between the Council and the affected field forces. The help of all those involved in various ways is acknowledged, especially that of the several district materials engineers, who coordinated the Council's involvement and subsequently secured various project data.

The project has extended over a considerable period of time and thereby has involved many people who have during this period been a part of the Concrete Section. In addition, the extensive petrographic assistance necessary has been provided by several people who during this period staffed that section. In a real sense the contribution of these people has been sufficient to warrant co-authorship. In this vein appreciation is expressed to Marvin Hilton, Harry Brown, and Ken McGhee, highway research engineers, and Dr. M. A. Ozol and Mrs. H. N. Walker, highway materials research analysts.

The extensive field tests were supervised by C. E. Giannini, and the similar petrography responsibility was discharged by Bobby Marshall, both materials technicians.

General direction was initially received from the late Dr. Tilton E. Shelburne, and subsequently from J. H. Dillard, from their position of state highway research engineer.

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APPENDICES

APPENDIX A

SCREEDING AND FINISHING TECHNIQUES

The screeding and finishing techniques used in the construction of concrete bridge decks differ markedly from those in the construction of concrete roadway pavements. Differences in the flexibilities of structural members, variations in the roadway width and span lengths from bridge to bridge, and occasionally on individual bridges, and a lack of working space outside the roadway width are a few of the factors which make screeding and finishing of bridge decks unique. Furthermore, in the construction of decks, the placing and the finishing of concrete are not day-to-day operations as they are in the construction of roadways, so the work crews on bridge decks are not usually as experienced nor as well organized as those on roadways. As a result, a number of factors are involved which reduce the chances of bridge decks being as smooth or as durable as concrete pavements.

Possibly because there were many variations in the size and configuration of bridge decks when these decks were placed, there were also many variations in the types of screeds used by different contractors. As a matter of economics, a contractor who infrequently constructed bridge decks might have a small manual type screed that could be adapted to almost any bridge deck; or he might simply improvise one to fit a specific occasion. Still another contractor might have used an elaborate mechanical type screed that could be adapted to various roadway widths or span lengths by adding or removing replaceable sections. The overall result was that a wide assortment of screeding devices of all types, sizes, and shapes were being used by the contractors to construct bridge decks. In 1968 manual screeding was prohibited by Virginia specifications so use of more elaborate mechanical screeding equipment has become general practice.

All types of screeding devices, whether mechanical or manual, must operate by rolling or sliding on screed rails or some other type of support. These rails are set either to the finished grade or at a predetermined height above the finished grade; plus an allowance for the anticipated dead load deflection. Since the location of the rails depends primarily upon the type and length or width of the screeding device to be used, they are located in various places.

Screeding devices are usually classified as being either longitudinal or transverse (on the basis of their orientation to the direction of traffic) and as being either mechanically or manually propelled. Some have mechanically oscillating screeding edges and others have vibrating edges. In this evaluation the many types of screeding methods were divided into the following categories:

1. Longitudinal Screeds
   - A. Manually operated types
   - B. Manually operated vibratory types
   - C. Mechanical, oscillating, full span length types

2. Transverse Screeds

   D. Manually operated types
   E. Manually operated vibratory types
   F. Mechanical oscillating types

A typical longitudinal manually operated screed is shown in Figure A-1. Although this particular one is a single truss type, the screeds in this category are constructed to various designs. In some instances they might consist only of a 2" x 6" board used to strike the concrete off to grade. A wide assortment of lengths can also be found in this type screed, but the significant factor is the longitudinal manner in which it is used. All screeds of this type are manually oscillated back and forth in a longitudinal direction while simultaneously being pulled across the width of the bridge deck roadway. The main variation in this operation is the location of the screed rails. In the situation shown in Figure A-1, the rails are set flush with the final grade and run transversely across the width of the deck at the midpoint of the span. Current specifications require that the screed rails must be located above the finished surface. In other cases the rails might be set at the 1/4 points, 1/3 points, or at other fractional portions of the span length which best accommodate the screed being used. After the screeding operation is completed, the rails are removed, and the remaining voids filled in with concrete and smoothed over. The entire deck surface is also finished by some type of floating operation immediately following the screeding.

The longitudinal manually operated vibratory type screed resembles the type just discussed in most respects, except that the screeding edge is not normally supported by a truss. It is usually a simple beam-like device vibrated by an engine mounted on the top side of the beam. Its general appearance is similar to that of the screed to be shown in Figure A-3, except it is, of course, used in a longitudinal position on the bridge deck. These type screeds are not normally oscillated in the longitudinal direction since the vibrations set up by the engine tend to consolidate the concrete and produce an even textured surface. They are, however, pulled across the width of the span manually, or in some cases, advanced by a reel and cable. The screed rails are placed in positions on the bridge deck similar to those described for the nonvibratory screed. Shorter spacing intervals are used more frequently, however, because the screed itself is usually shorter in length than the truss type screed. After the screeding operation, the rails are removed and floating procedures follow immediately behind the screeding operation, as described above. The longitudinal mechanical, oscillating, full span length type screed has been used in Virginia on spans up to approximately 100 feet in length. In Figure A-2 a screed of this type is shown being used to screed a 90-foot span. It is constructed with double trusses spaced approximately 18 inches apart to provide lateral stiffness, and an adjustable channel attached to the lower portion of the truss serves as the screeding edge. The screed oscillates in a longitudinal direction while simultaneously creeping across the roadway width. It is mechanically actuated by a 3/4 horsepower electric drill motor attached in a bracket at one end of the screed. Two wheels at each end of the screed, which can be raised and lowered by hydraulic hand operated jacks, are used to roll the screed back into position for additional passes. Unless this screed is used on excessively long spans, the only screed rails used are mounted at the bulkhead forms at the ends of the span. Consequently, the rails are not subject to change in elevation due to dead load deflection such as that encountered with rails mounted at intermediate points in the span. The final finished grade can thus be set directly to the adjustable screeding edges at desired intervals along the length of the span and it remains independent of the dead load beam deflections. Screeding is followed by a full span length longitudinal belting operation, which is best accomplished just prior to the initial set of the concrete.

The transverse manually operated type screeds are similar to the manually operated longitudinal screed shown in Figure A-1 and are found in just as many assortments of sizes and shapes. These screeds rest on the deck in a transverse position, however, and are usually found in lengths sufficient to cover either half or the full width of a bridge deck. They are manually oscillated back and forth transversely and are simultaneously pulled down the length of the span on screed rails set at or near the curb line.

The transverse, manually operated, vibratory type screeds are similar to those of the longitudinal vibratory type, except that they are rotated through 90° and are used in a transverse position. Like the nonvibratory transverse types, they usually cover either half or all of the span width, and the screed rails are placed longitudinally down the span in similar positions. Figure A-3 shows one of the common type vibratory screeds that could be classified as semi-mechanical, since it can be advanced down the span by reels and cables mounted at each end.

For the last two methods described, the surface is sometimes longitudinally floated and/or smoothed with a bull float immediately, see Figure A-4.

The transverse mechanical oscillating screed shown in Figure A-5 incorporates a box type truss design to support two screeding edges that can be adjusted to fit any specified roadway crown. The screeding edges oscillate transversely and the machine advances down the span under its own power — though some of the earlier models are propelled by a reel and cable. The box truss is of a telescopic design and can be adjusted to fit various widths of roadway. The machine rolls along longitudinal rails placed near the outside edges of the bridge deck roadway. After the screed makes several passes over a given area, the rail sections are removed, the voids filled in, and the general floating of the surface performed with a bull float.

Figure A-1. Longitudinal manually operated single truss type screed.

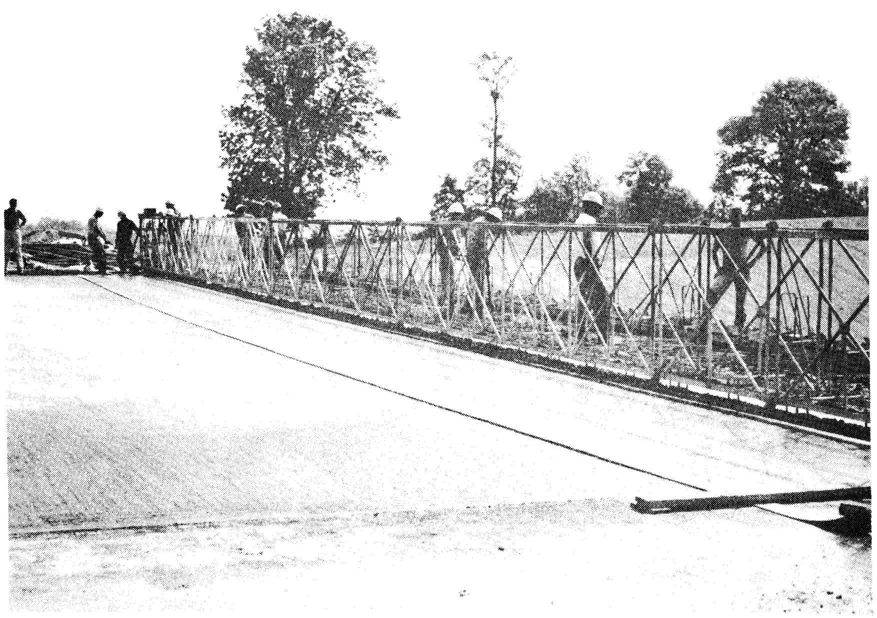


Figure A-2. View of a full span length mechanical longitudinal type screeding machine being used on a 90 foot span. A longitudinal belt used to finish the surface lies to the left of the screed.

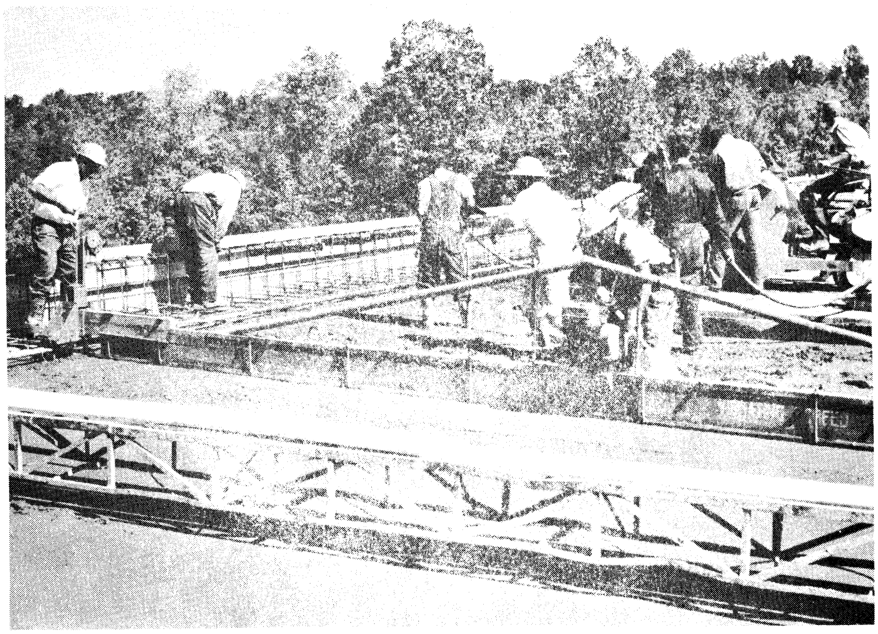


Figure A-3. Transverse vibratory type screed which covers the full width of the deck and is propelled by reel and cable. Note work bridge in foreground.

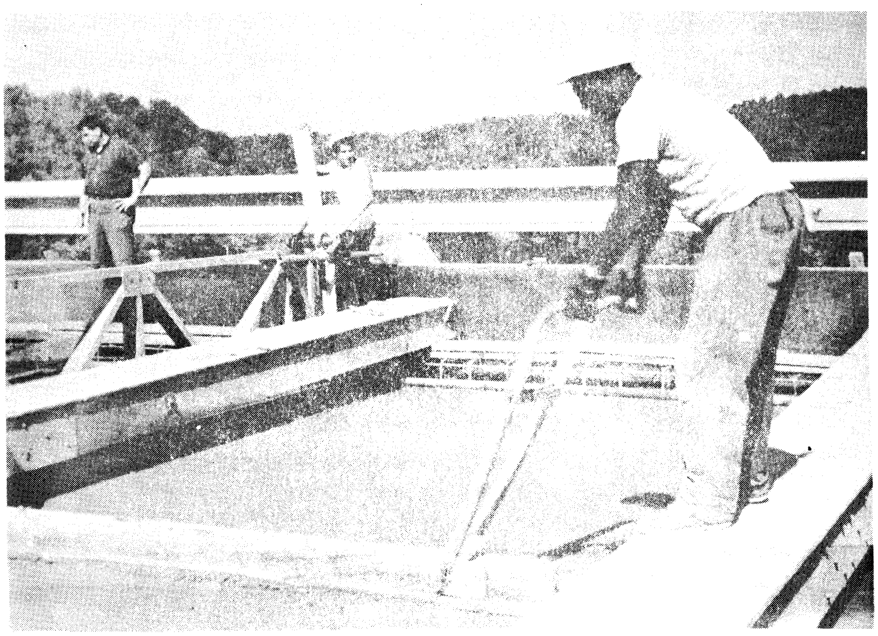


Figure A-4. Longitudinal floating of a bridge deck.

Figure A-5. Mechanical transverse oscillating type screed.

APPENDIX B

CHARACTERISTICS OF STUDY BRIDGES

For the purposes of classifying structural types during a 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 are 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, non-composite 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.

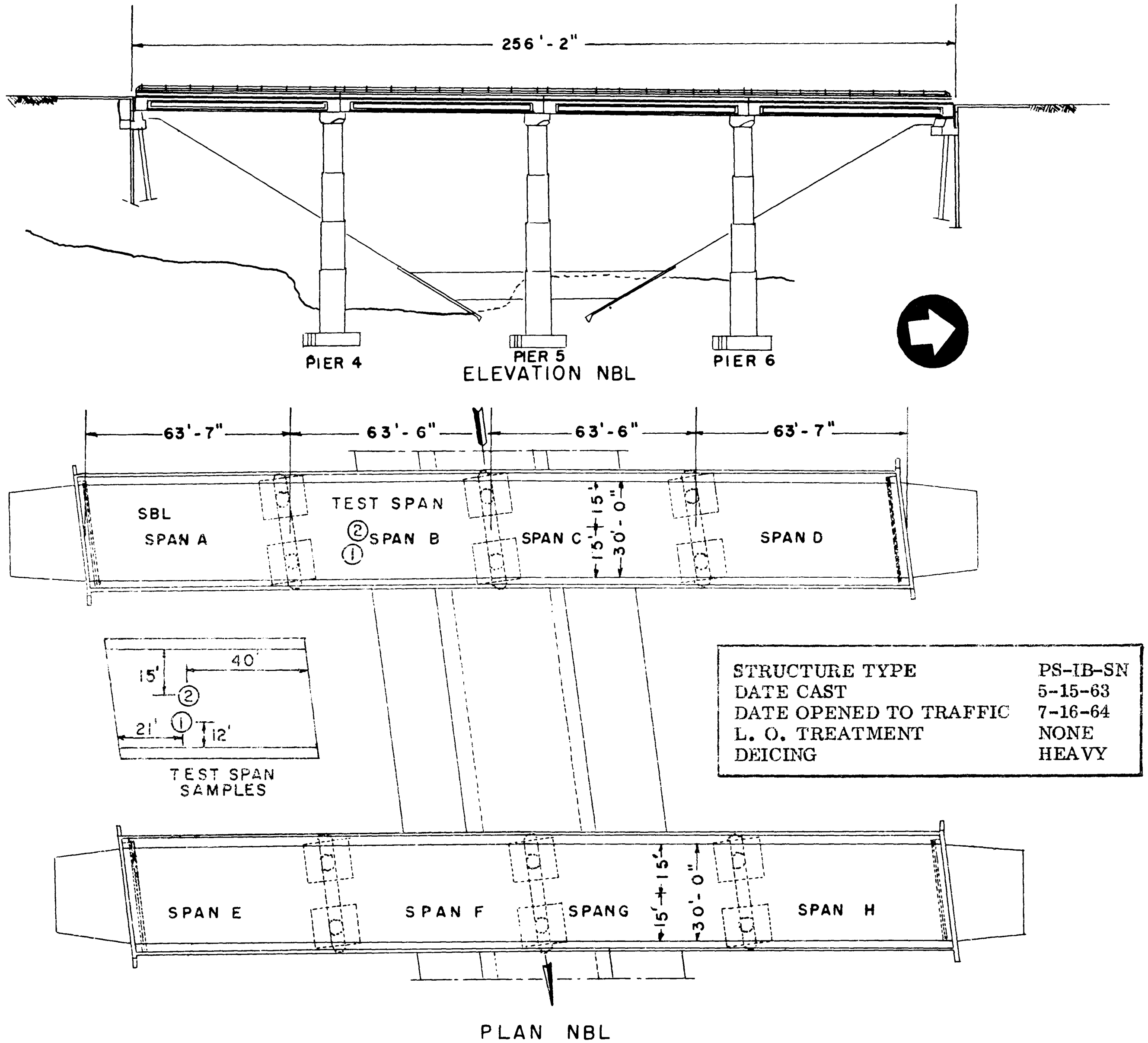
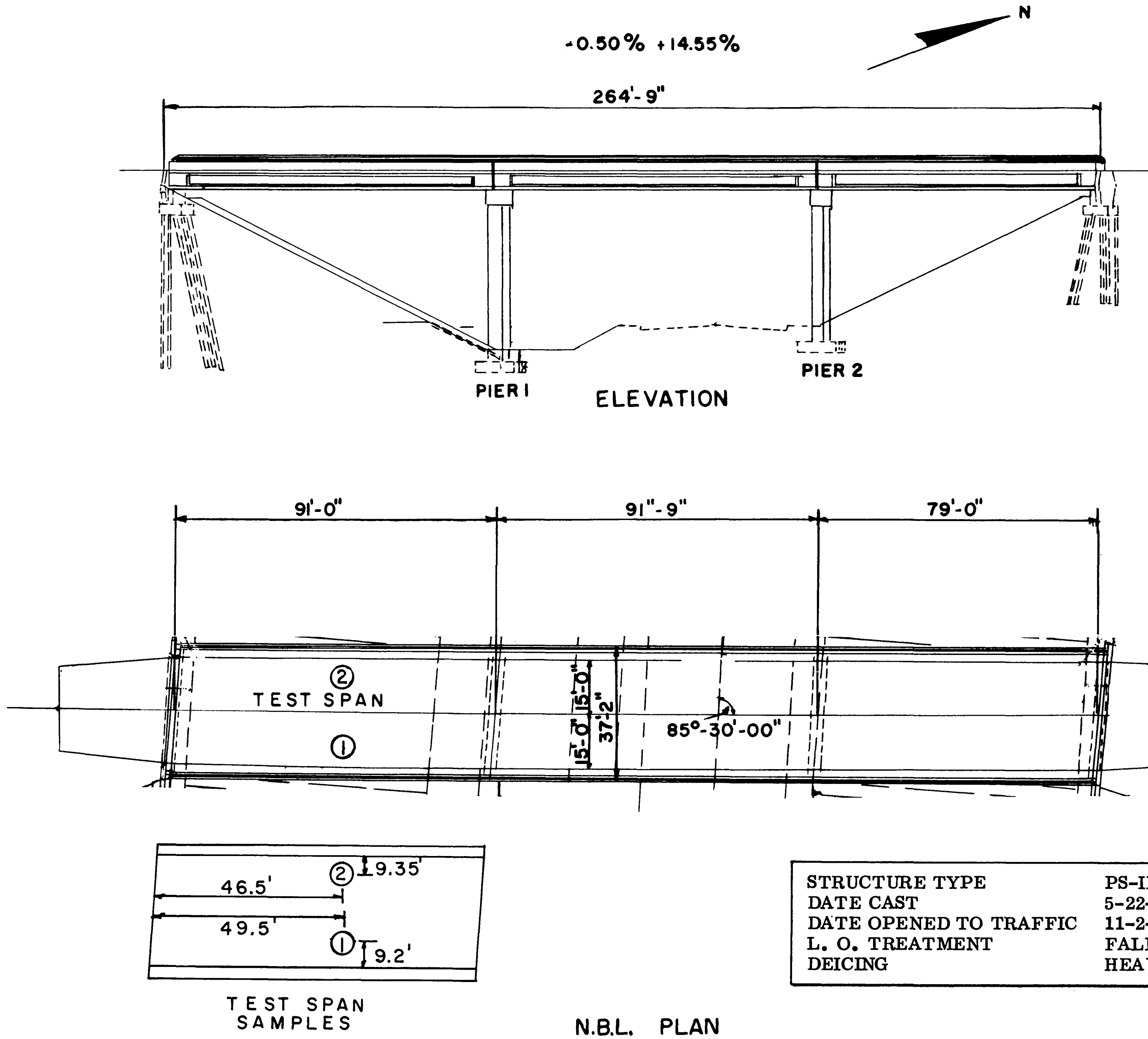
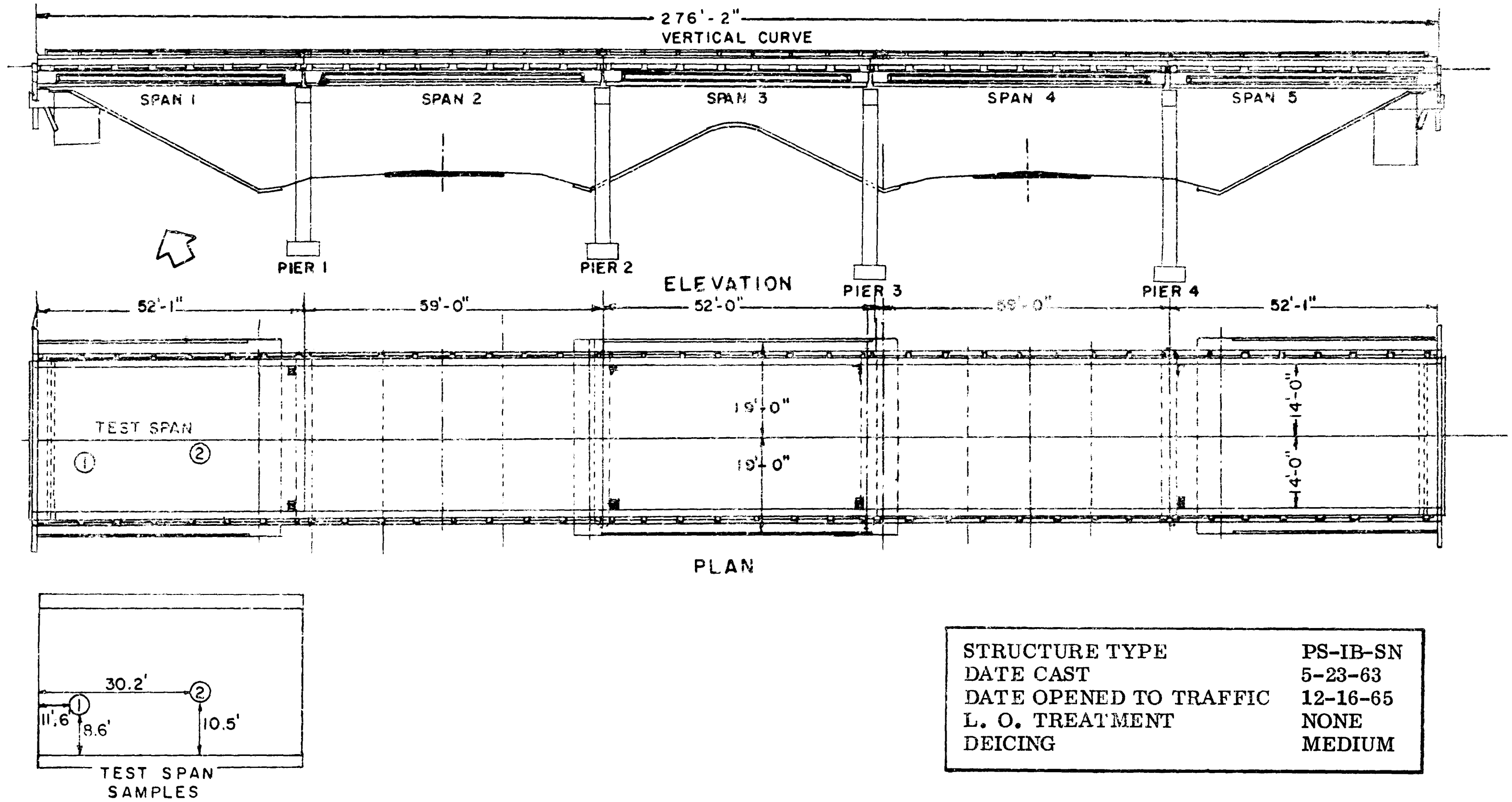


Figure B-1. Plan for bridge #1; also shown are the locations of test samples.



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

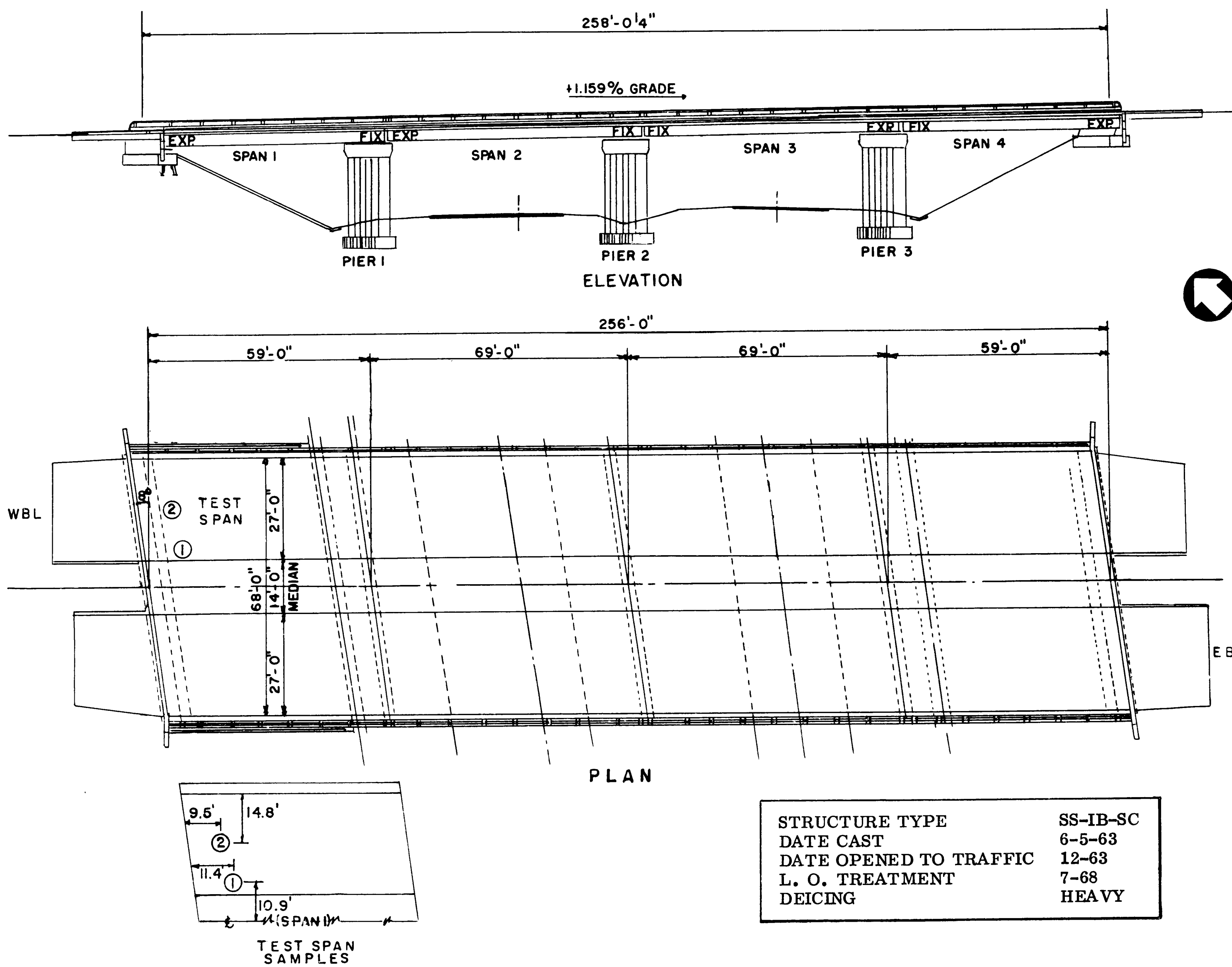
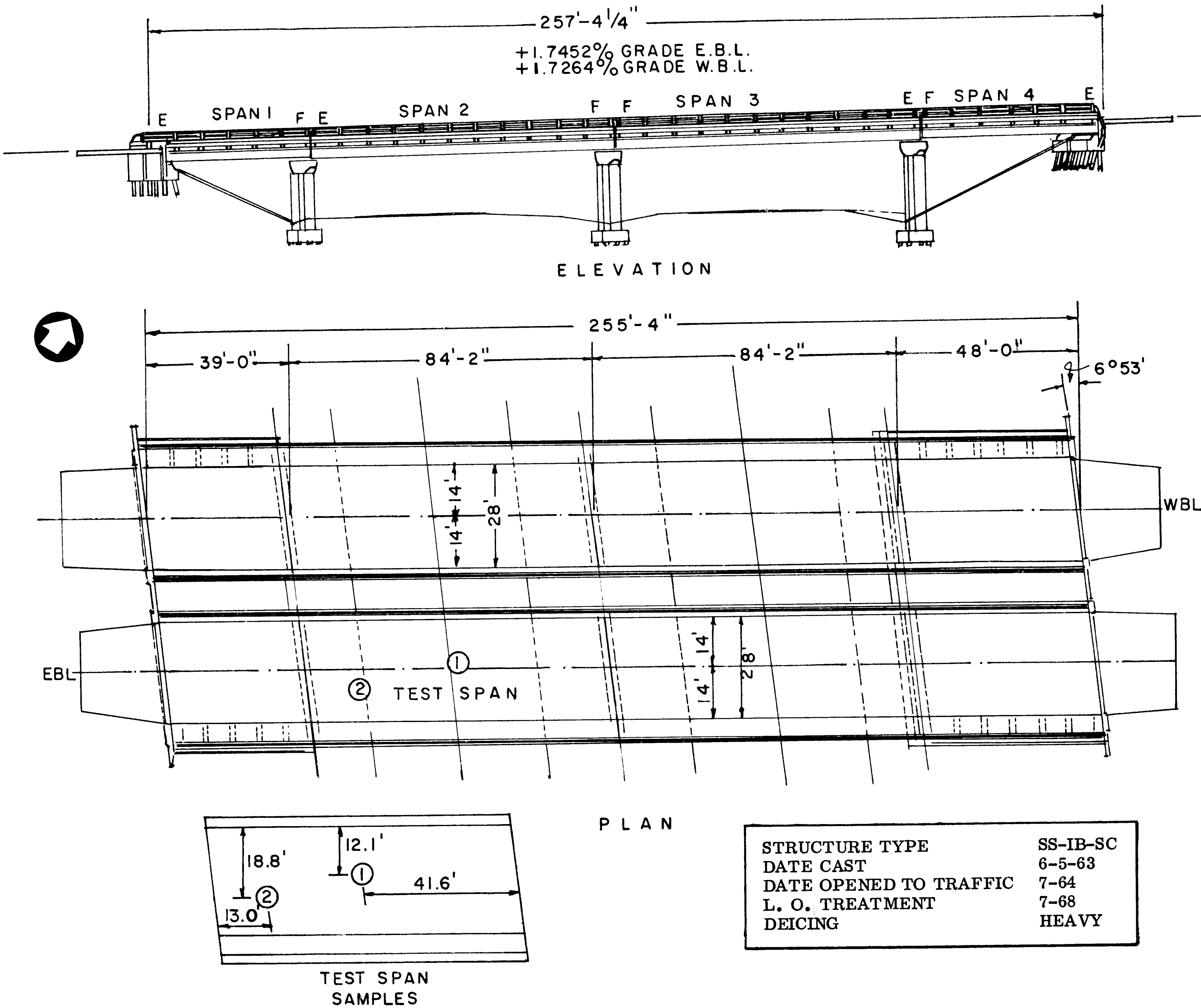
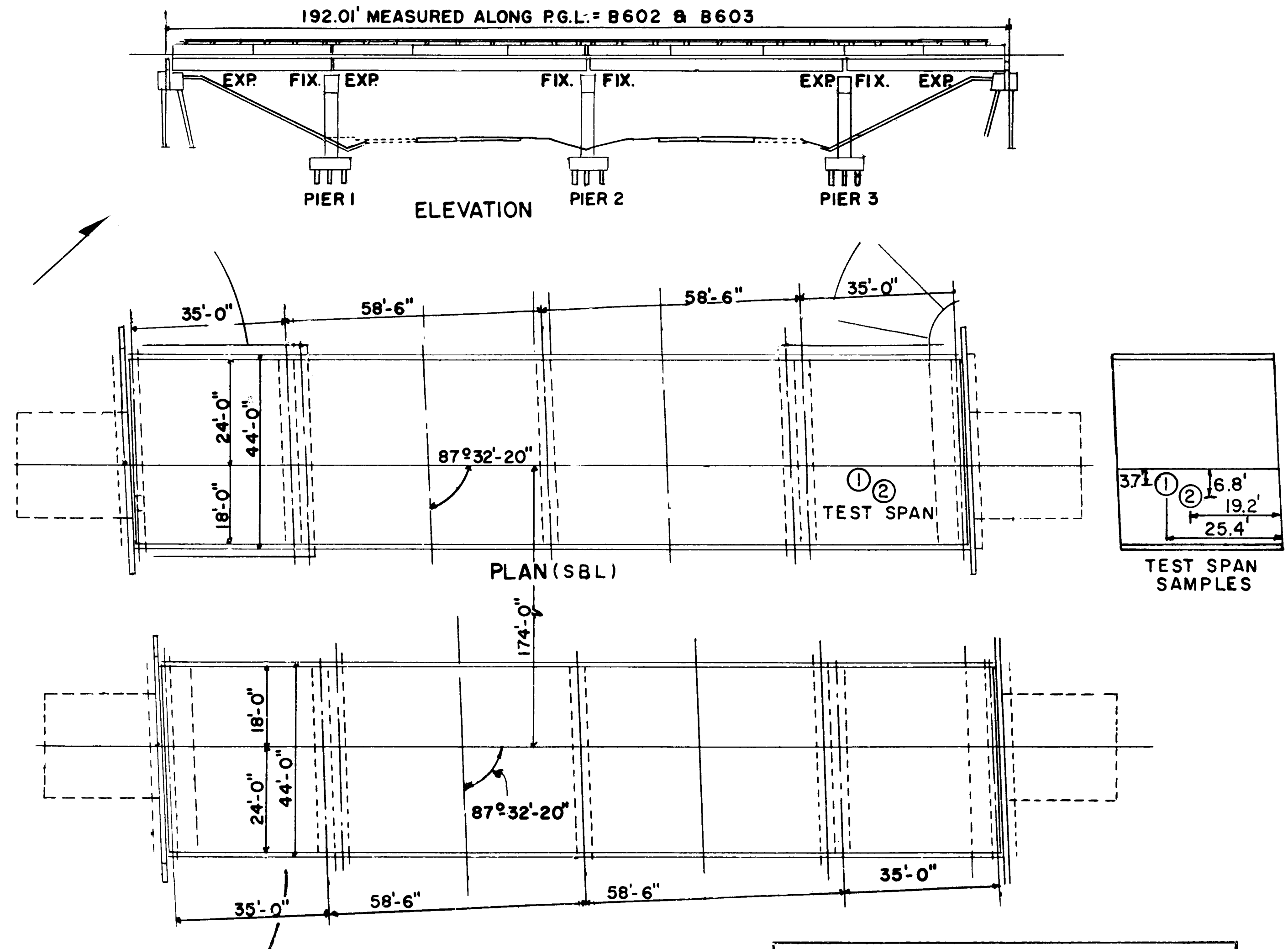


Figure B-4. Plan for bridge #4; also shown are the locations of test samples.



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

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

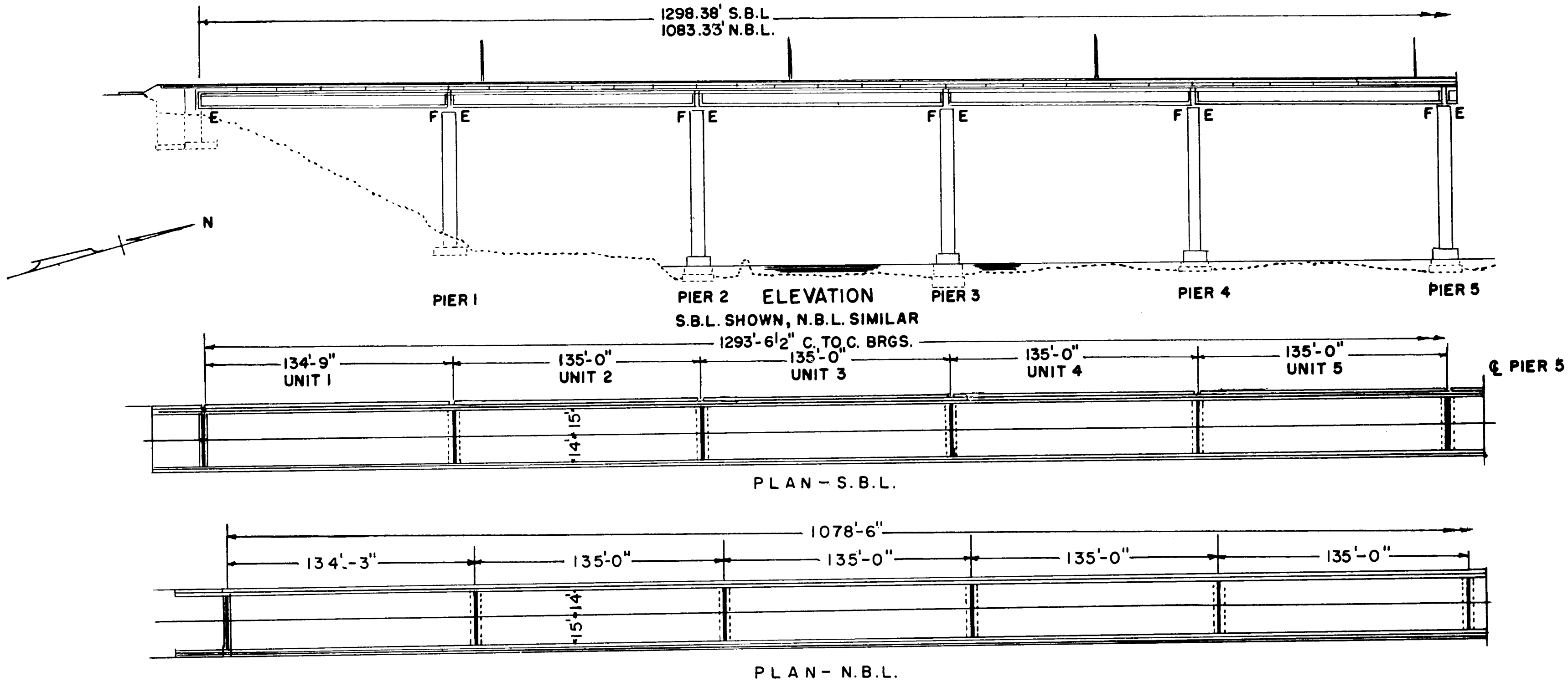
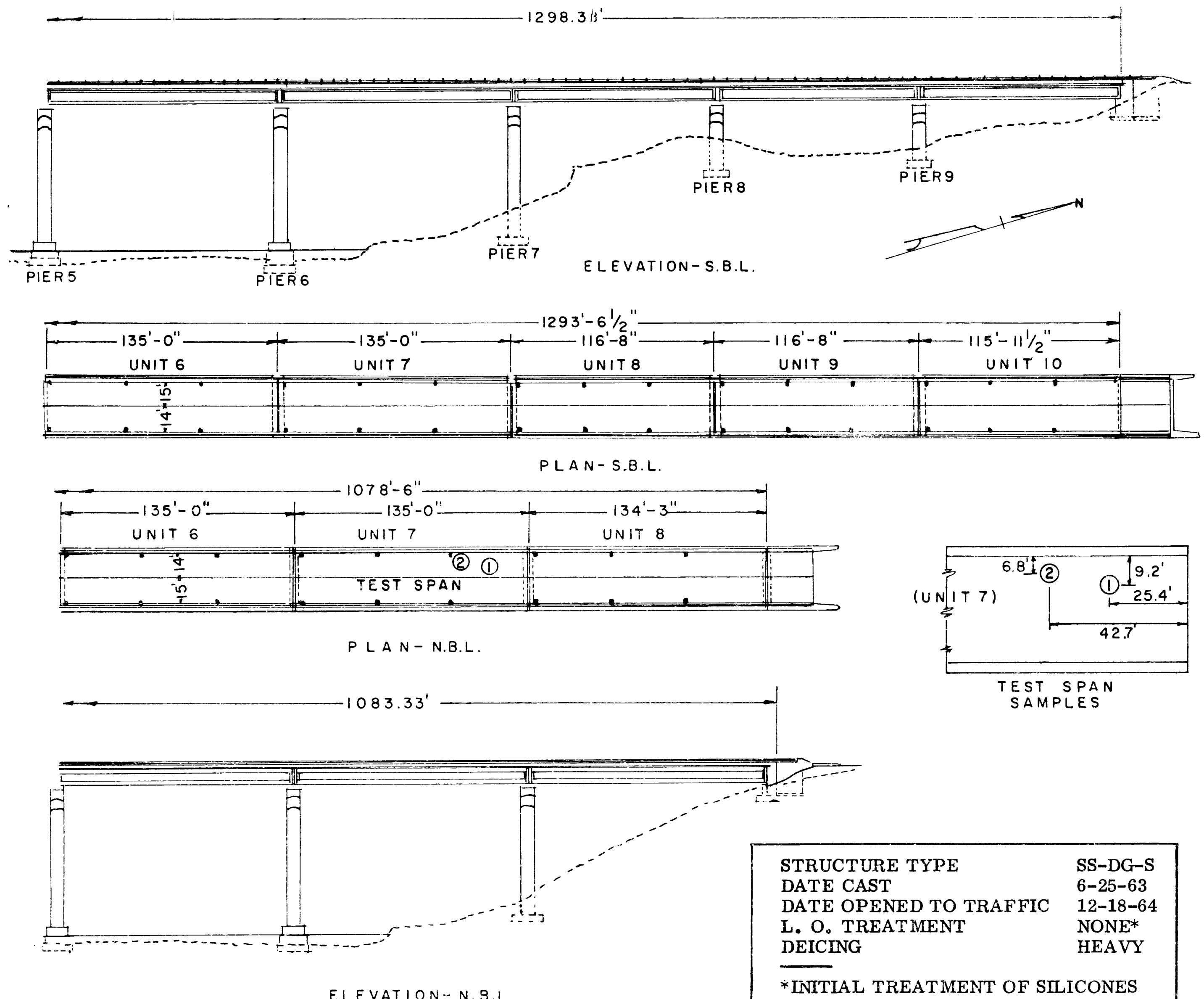


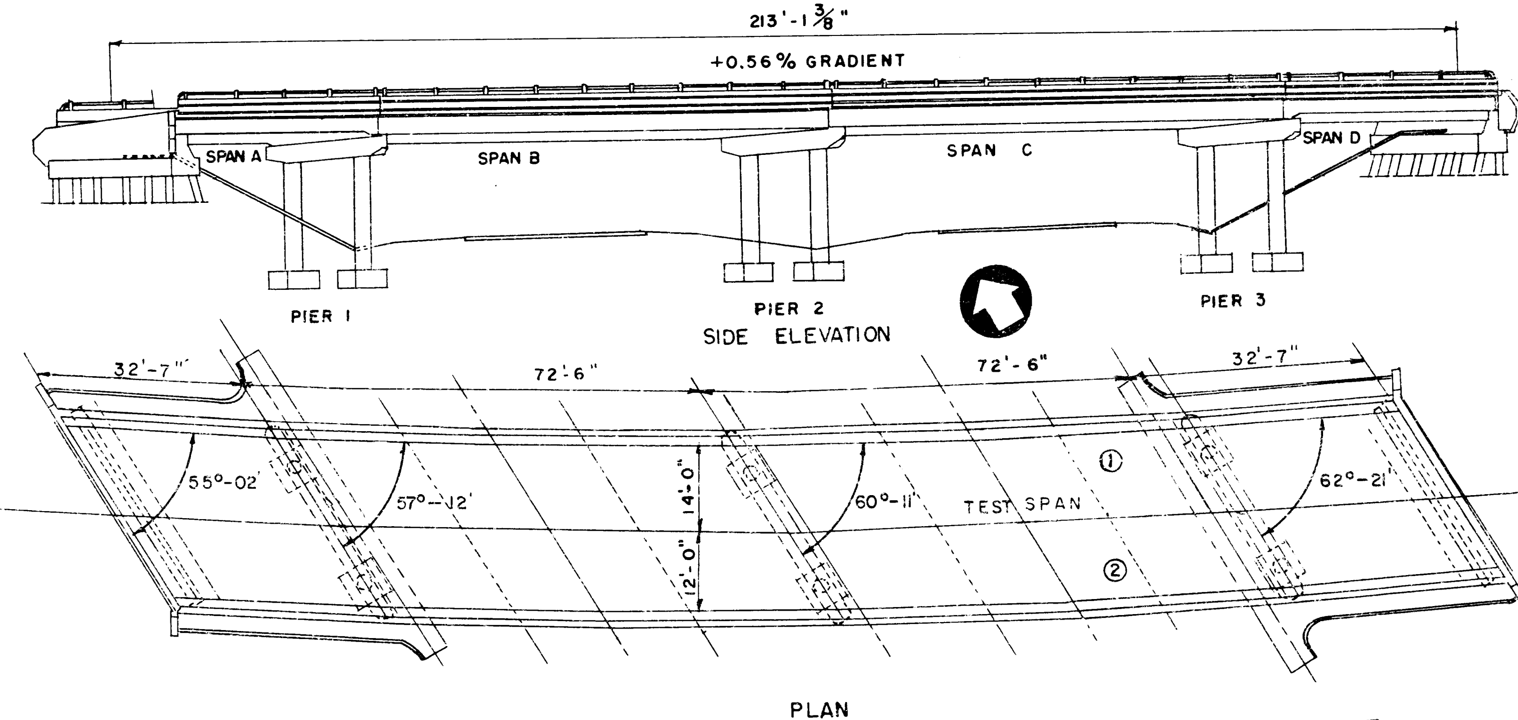
Figure B-7(a). Plan for bridge #7 (southern section); also shown are the locations of test samples.



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 B-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

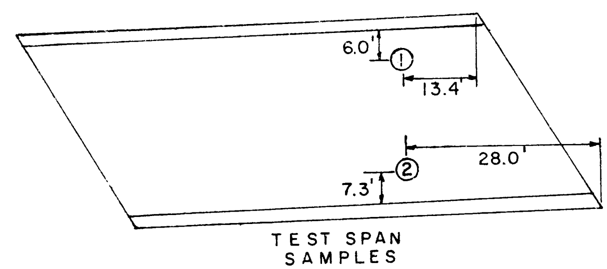


Figure B-8. Plan for bridge #8; also shown are the locations of test samples.

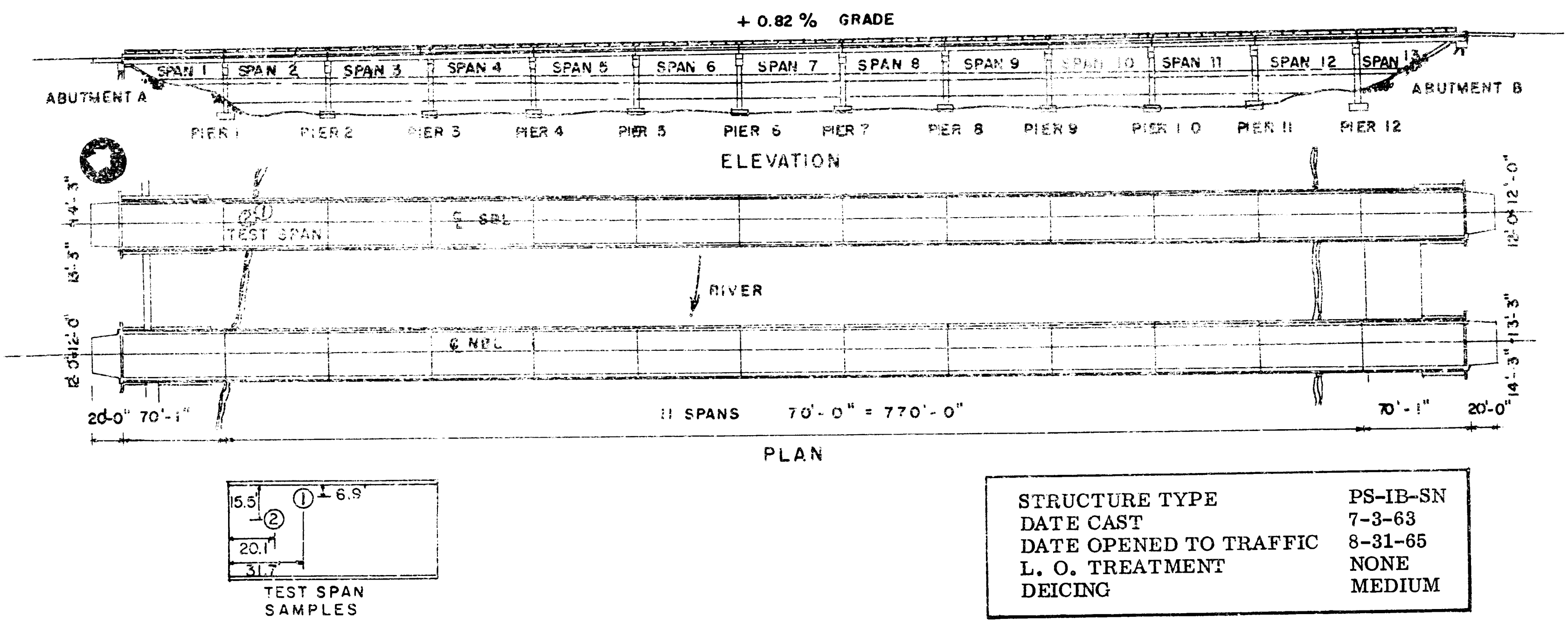


Figure B-9. Plan for bridge #9; also shown are the locations of test samples.

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

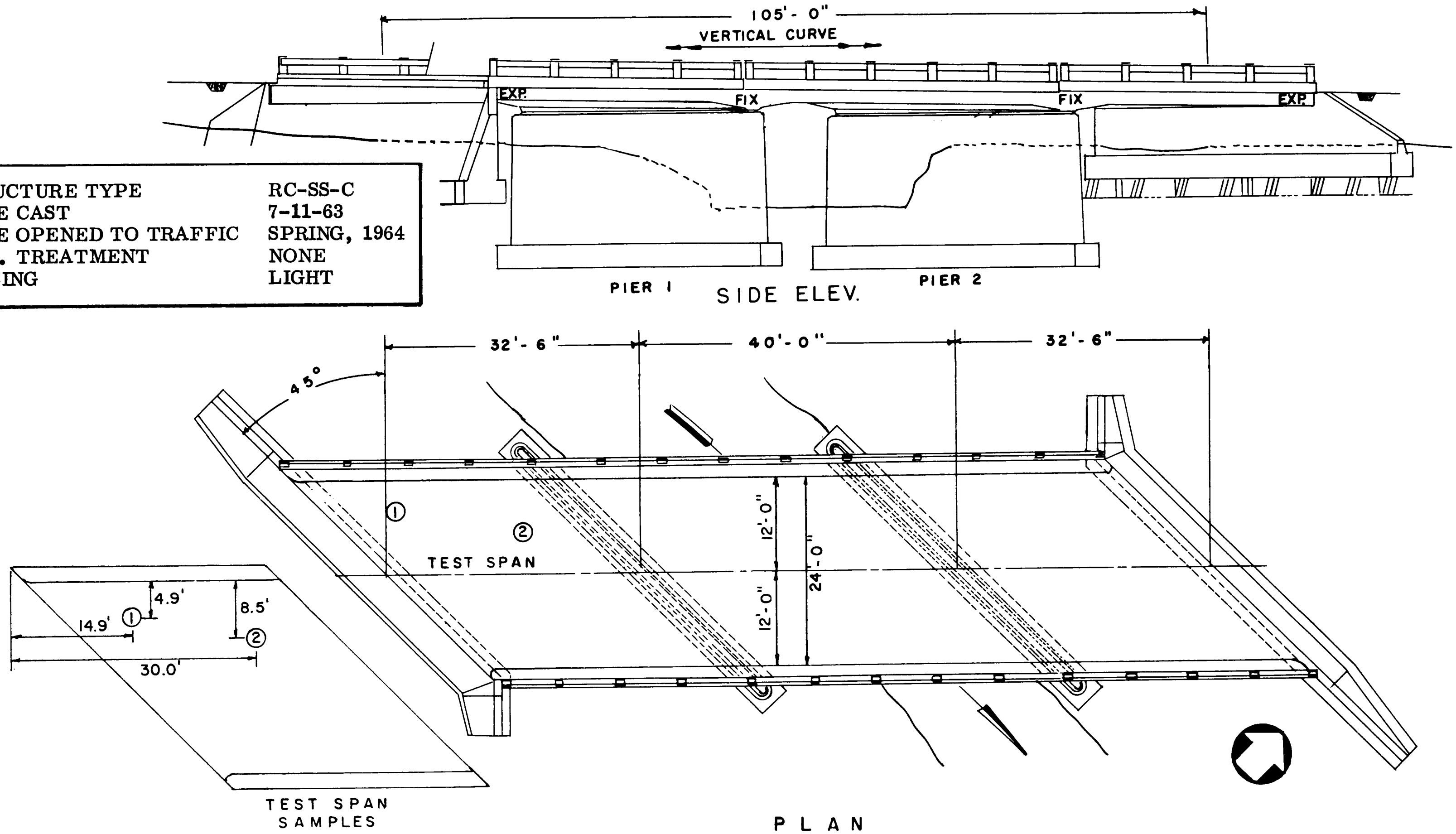
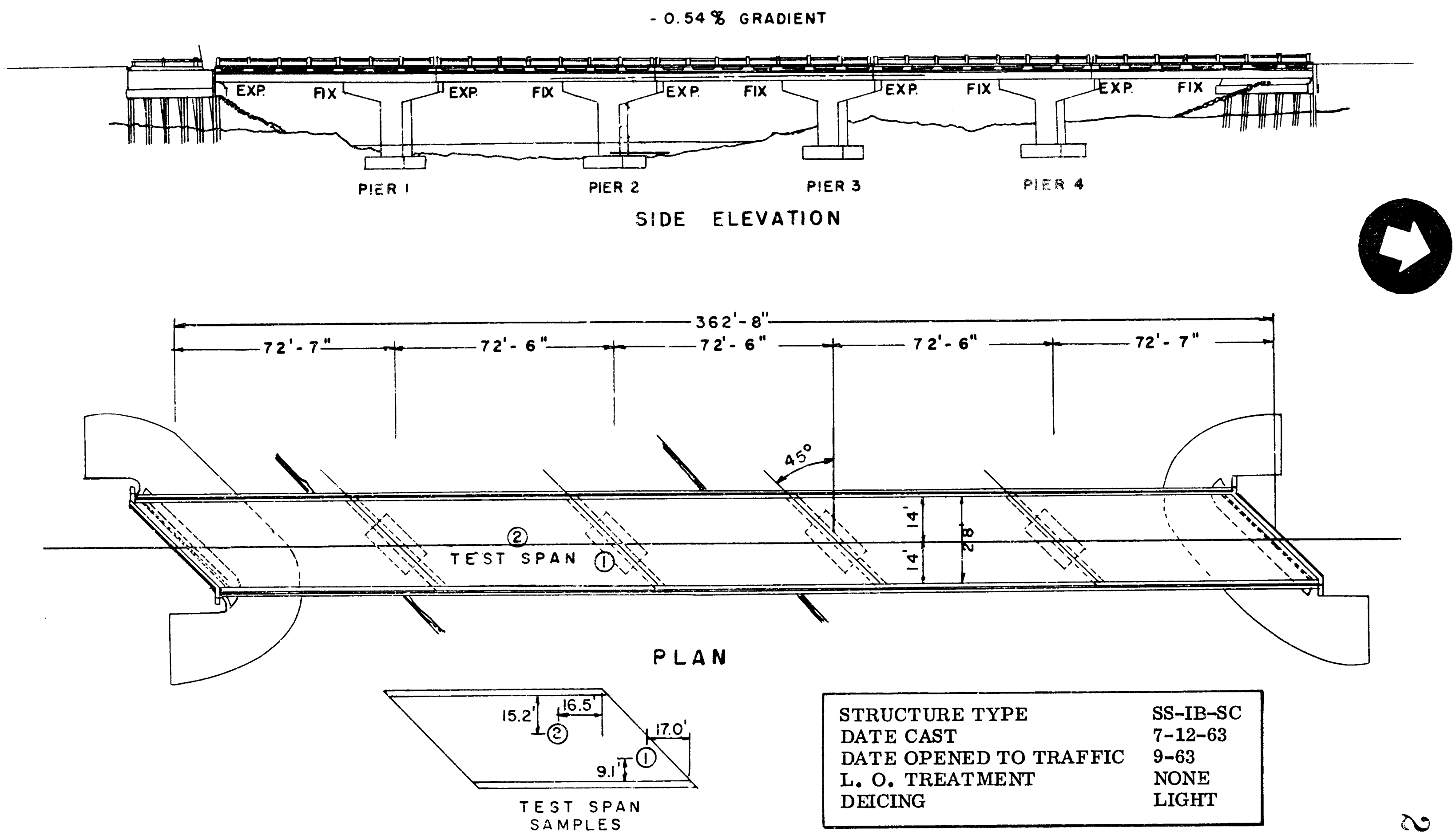
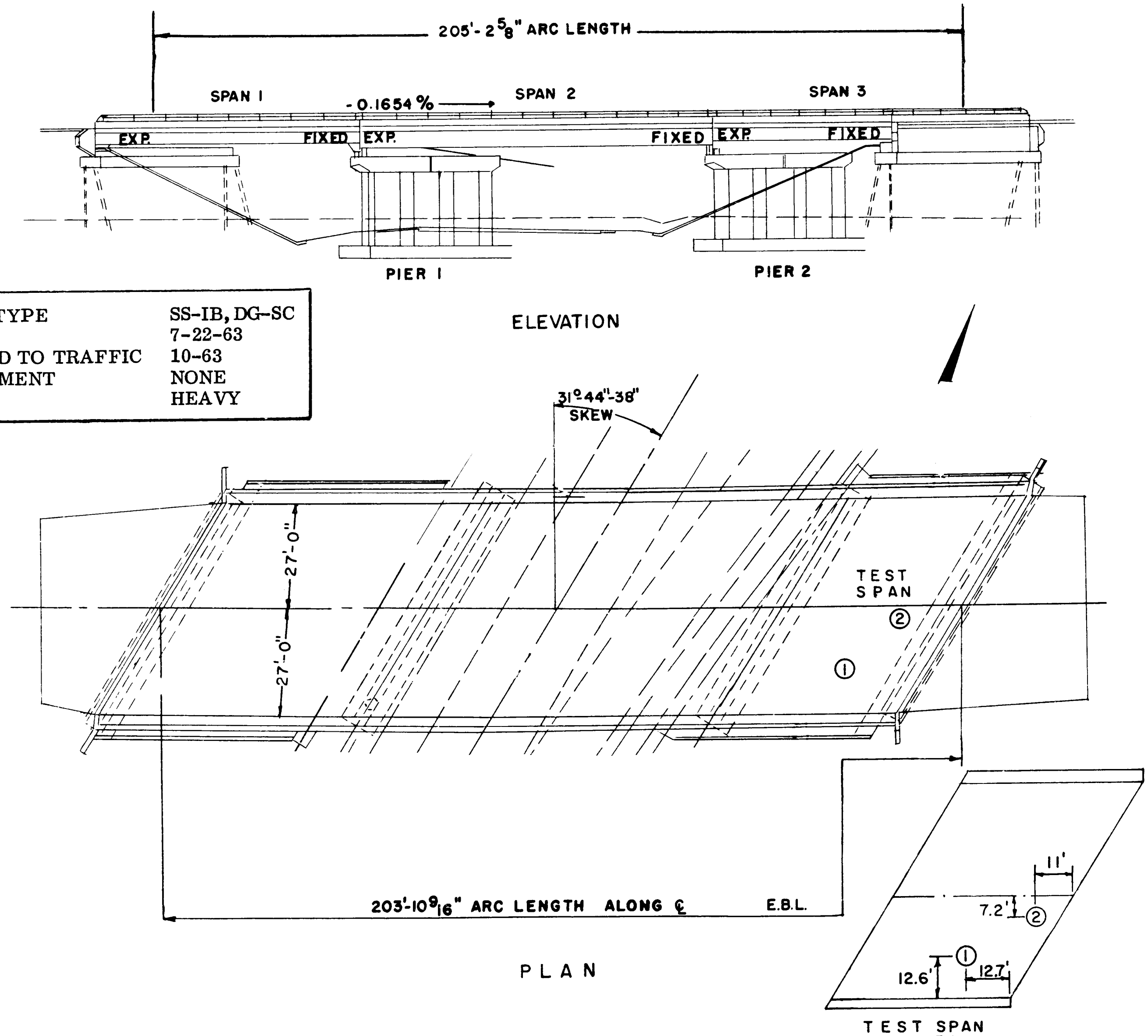


Figure B-10. Plan for bridge #10; also shown are the locations of test samples.



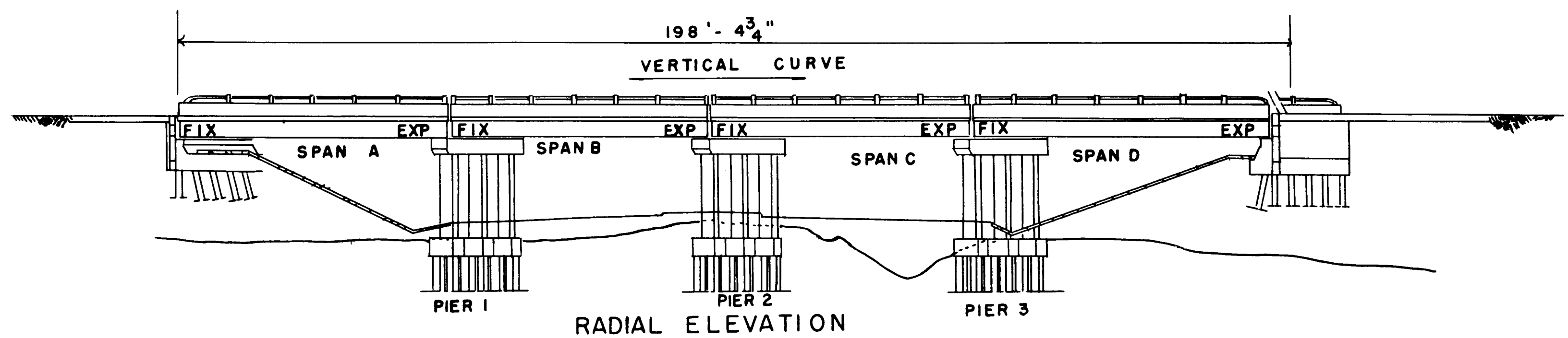
STRUCTURE TYPE	SS-IB-SC
DATE CAST	7-12-63
DATE OPENED TO TRAFFIC	9-63
L. O. TREATMENT	NONE
DEICING	LIGHT

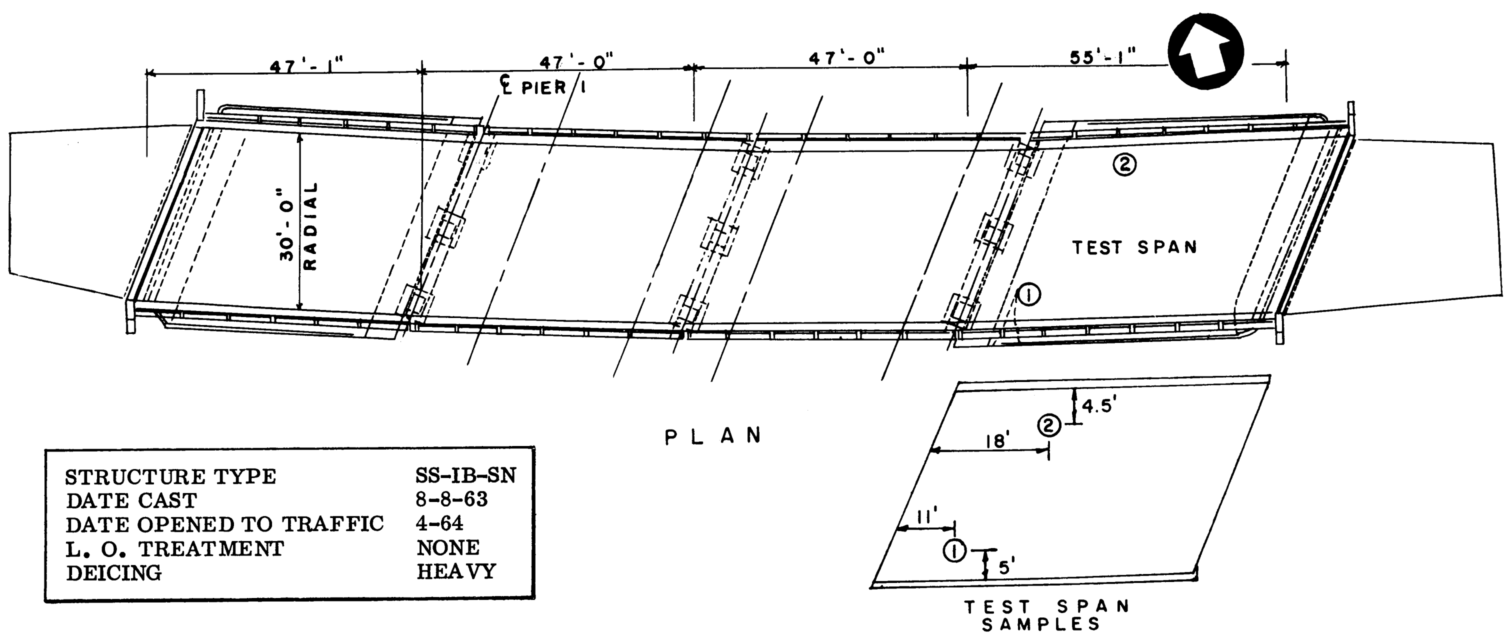
Figure B-11. Plan for bridge #11; also shown are the locations of test samples.



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

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

2718

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

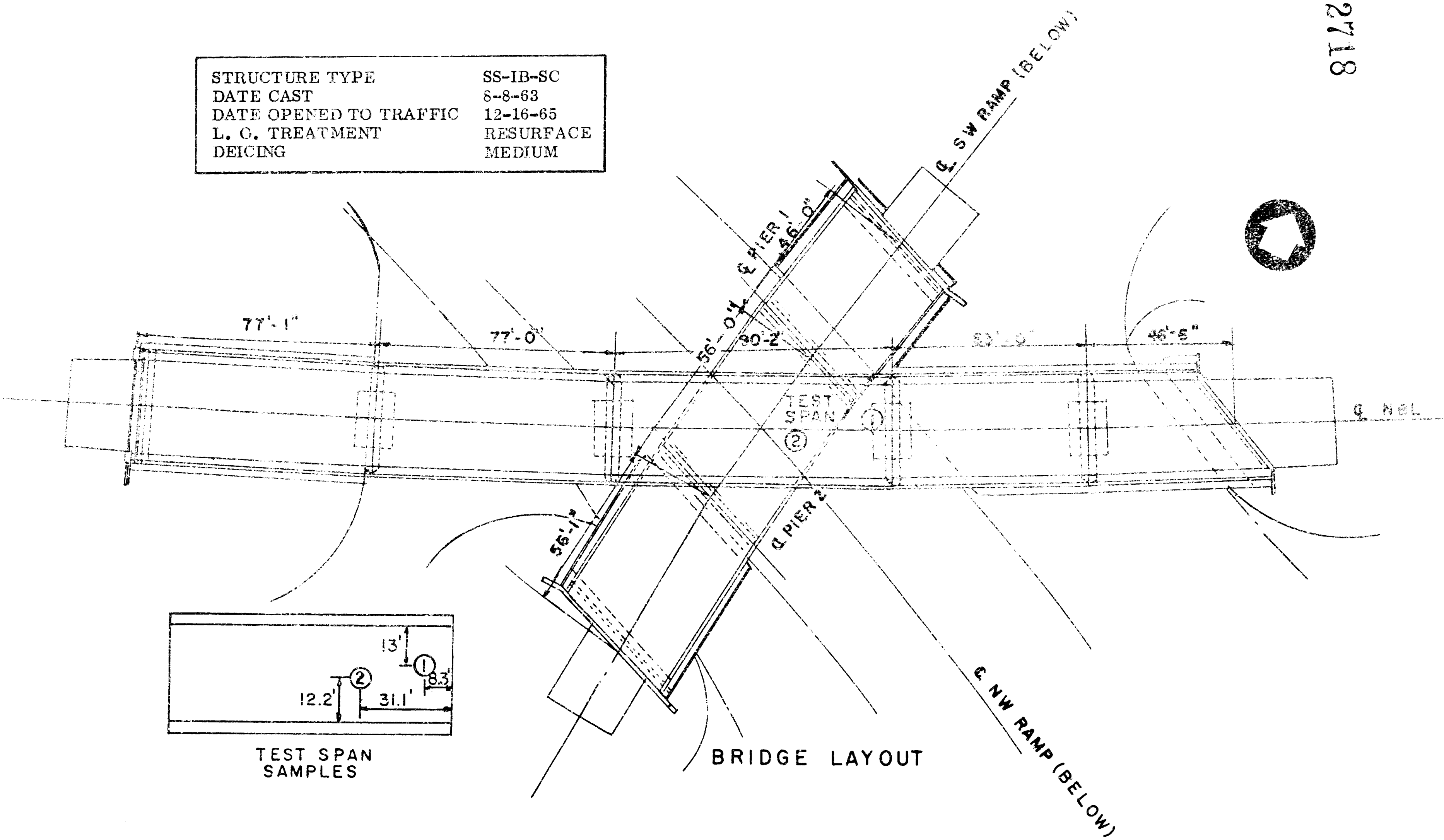
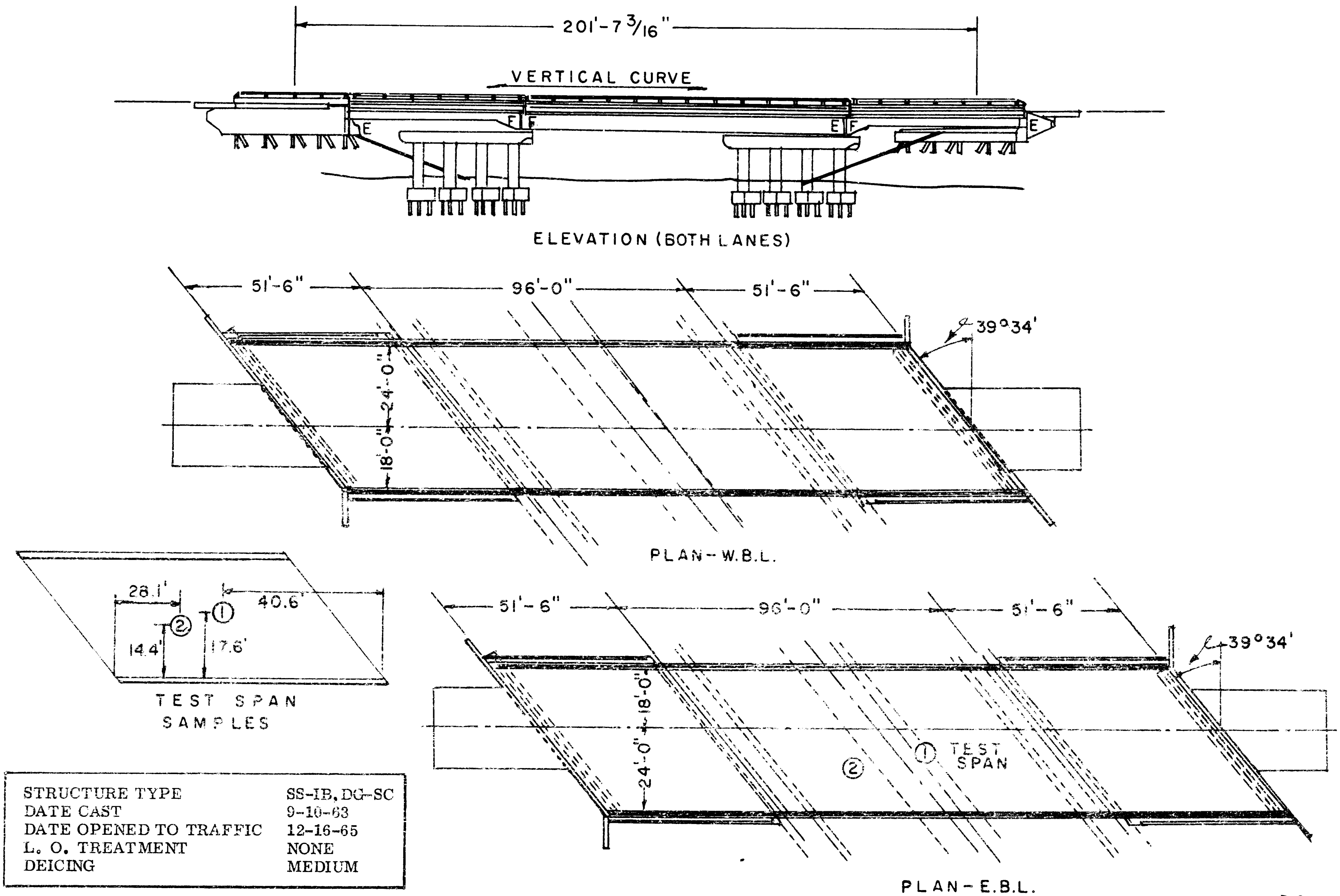


Figure B-14. Plan for bridge #14; also shown are the locations of test samples.



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

2720

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

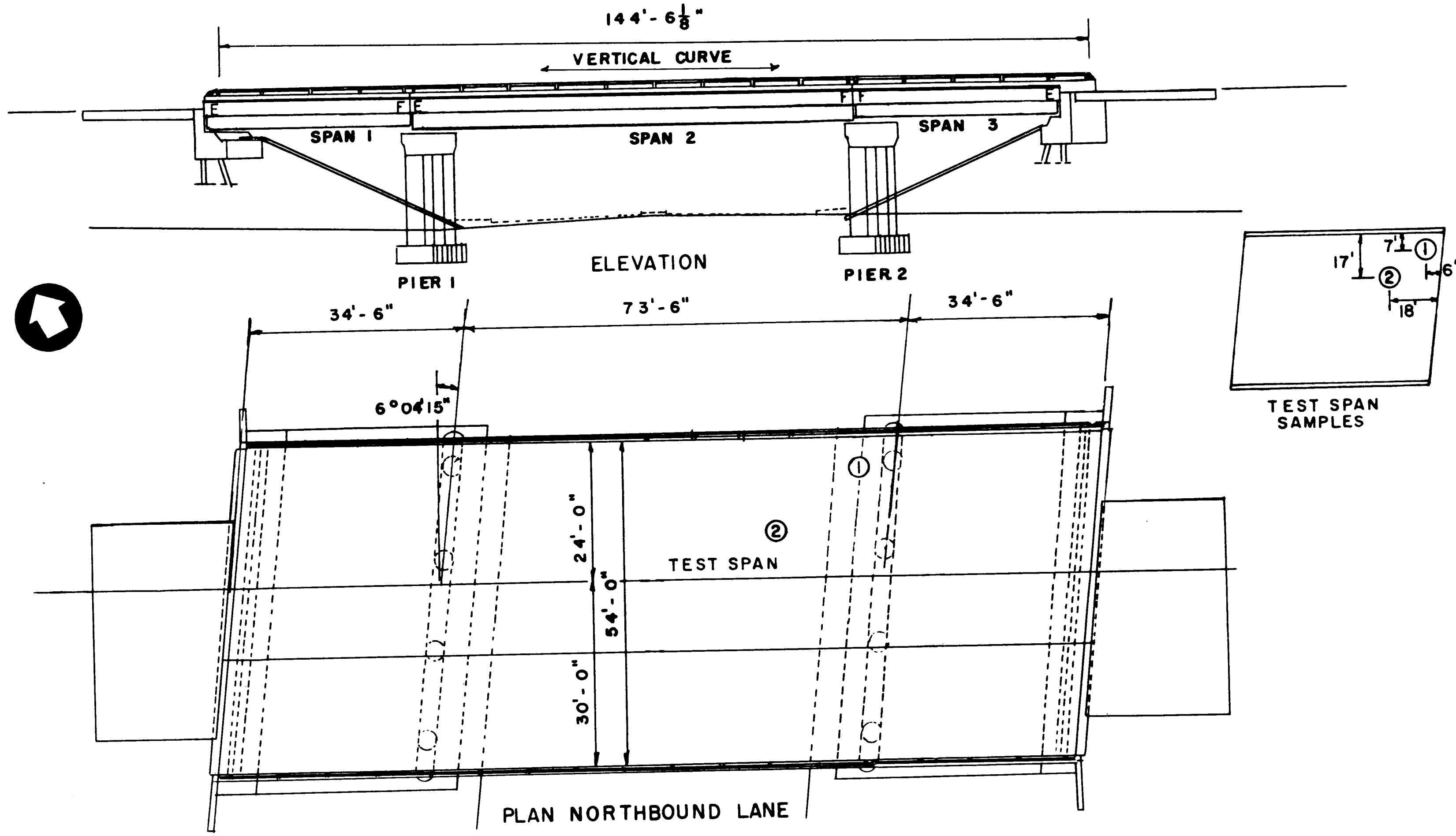
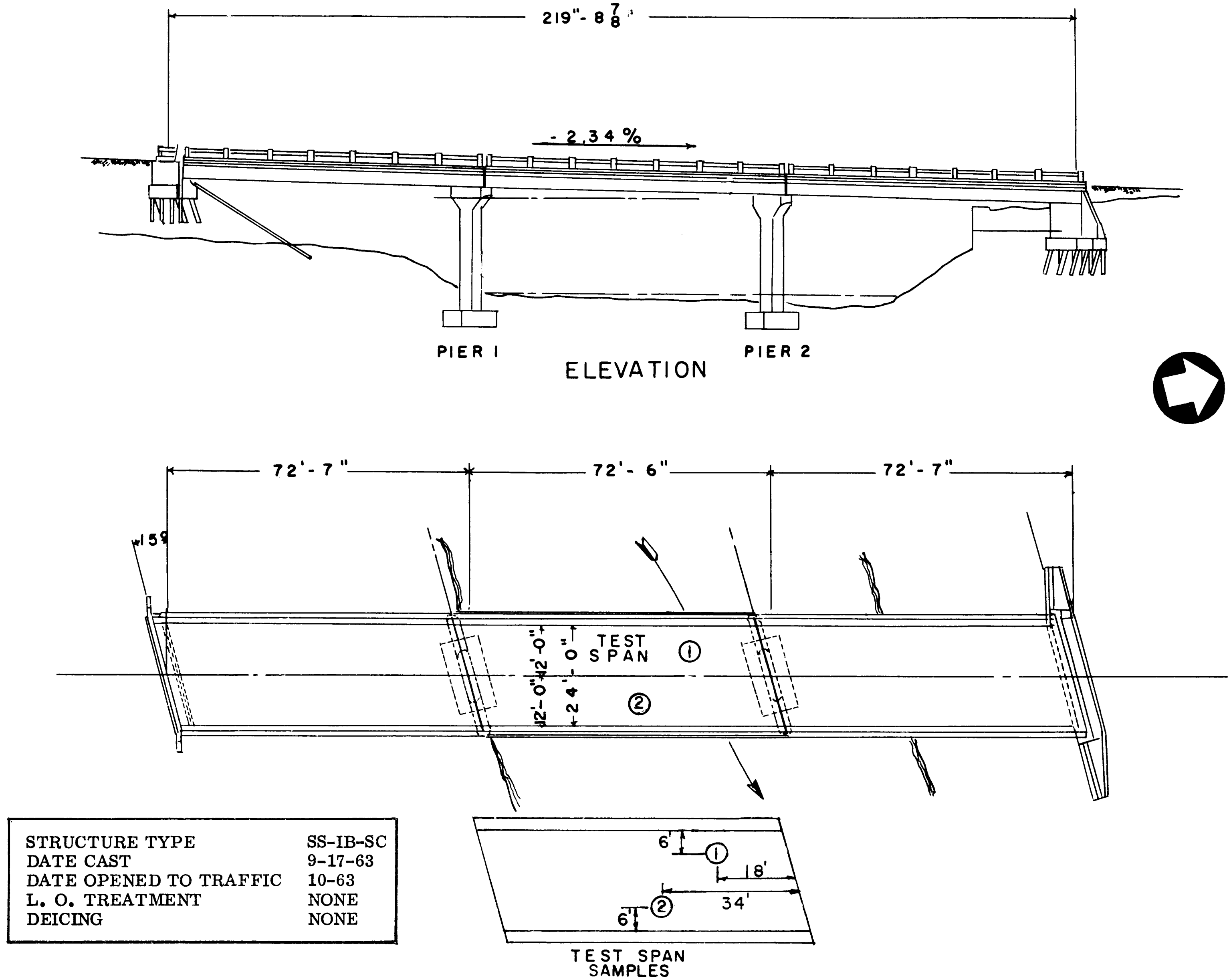


Figure B-16. Plan for bridge #16; also shown are the locations of test samples.



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

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

APPENDIX C

DEFECTS COMMONLY ENCOUNTERED ON BRIDGE DECKS

General

The riding surface of a bridge deck ideally should provide a continuation of the pavement segments which it connects. The surface should be free from characteristics or profile deviations which impart objectionable or unsafe riding qualities. The desirable qualities should persist with minimum maintenance throughout the projected service life of the structure.

Many decks remain smooth and free from surface deterioration and retain skid resistance for many years, attesting to satisfactory attention to the many details influencing such performance. When deficiencies do occur, they usually take one of the forms described in this appendix.

Roughness

Roughness can be periodic, varying in wavelength, or it may occur as discrete discontinuities. Excessive sag or camber are deficiencies which cause long wavelength roughness which may exist when the deck is new. Roughness with short wavelength, or "washboarding", can appear early and result from construction practices, or it can develop subsequently with surface deterioration. Such shortwave roughness may be periodic or random, depending upon its cause. Discontinuities at joints or near abutment backwalls result in sudden "bumps". Council research relating to roughness has been previously reported (Hilton 1968).

Cracking

Cracks may be classified according to their orientation in relation to the direction of traffic as longitudinal, transverse, diagonal, or random. In addition, the terms pattern cracking and crazing are used to refer to characteristic defects (ACI Committee 201 1968). The severity of cracking is conventionally expressed qualitatively as fine, medium and wide. Standards established by the ACI define cracking severity as follows:

fine — generally less than 1 mm,

medium — between 1 mm and 2 mm,

wide — over 2 mm.

Examples are shown in Figures C-1 through C-4.

Longitudinal cracking is most prevalent as "reflective" cracks in thin concrete wearing courses over longitudinal joints of precast-prestressed box girder spans or in areas where resistance to subsidence is offered by longitudinal reinforcement, void tubes, or other obstructions.

Figure C-1. Transverse cracking.

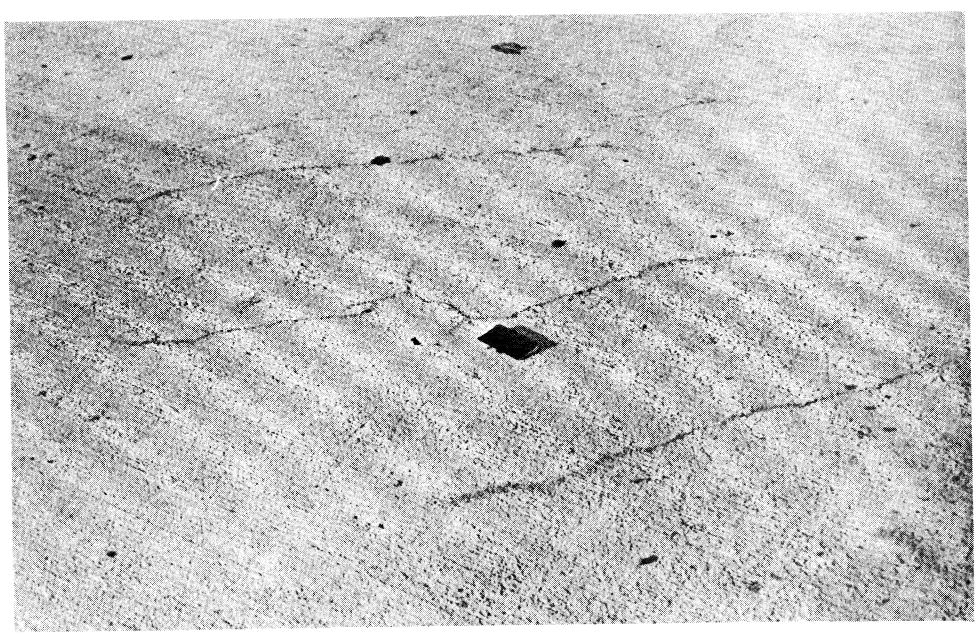


Figure C-2. Diagonal cracking.

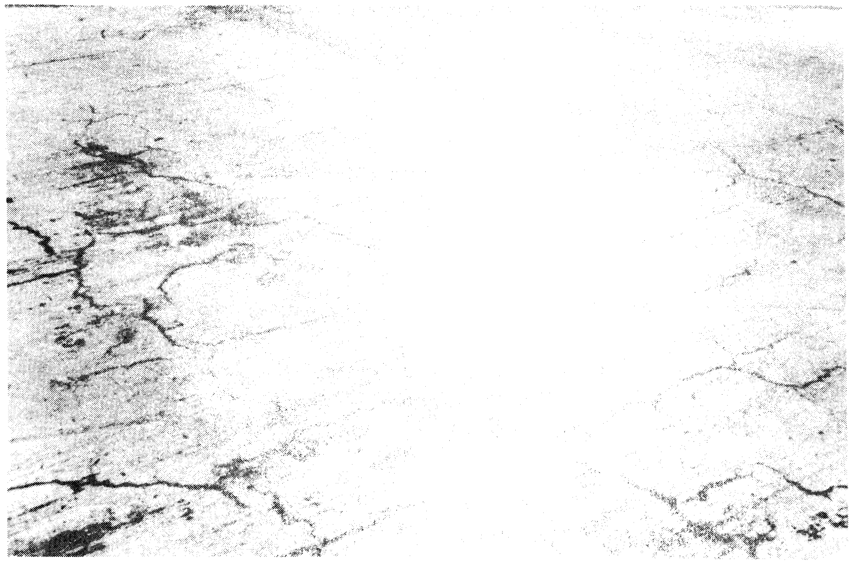


Figure C-3. Random cracking.

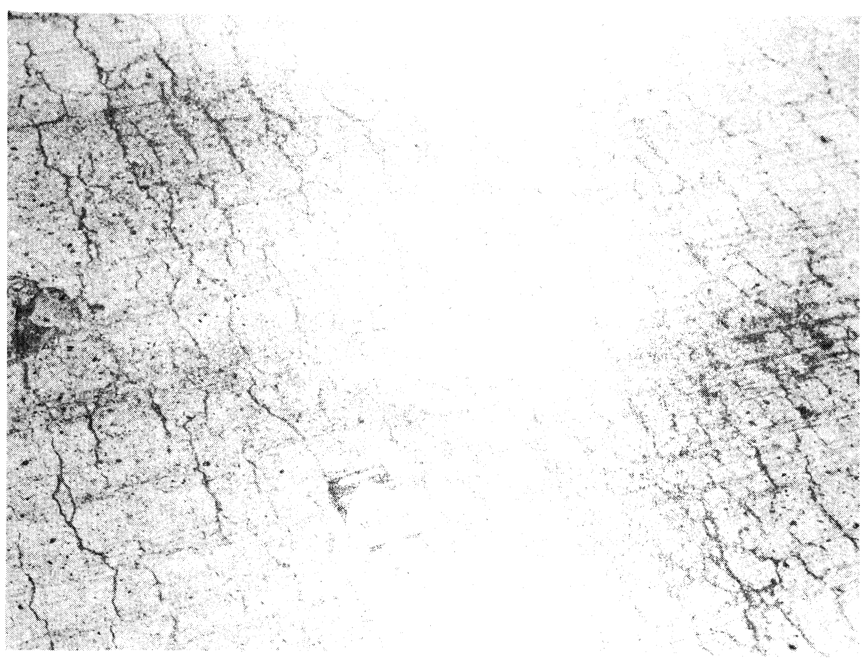


Figure C-4. Pattern cracking.

Diagonal cracking occurs most often in the acute angle corner near abutments of skewed bridges or over single column piers of concrete box girder, deck girder or hollow slab bridges.

No one factor can be singled out as the cause of transverse cracking. Among the more important factors are (1) external and internal restraints on the early and long-term shrinkage of the slab, and (2) combinations of dead load and live load stresses in negative moment regions. In general, the observed crack patterns suggest that live load stresses play a relatively minor role in transverse cracking except in continuous spans.

Pattern and/or random cracking is usually shallow and may be related to early or long time drying. Such minor cracking is a common defect. Occasionally severe cases are encountered in which cracks conform to a pattern but extend through the slab. Under these conditions the probable causes are severe early drying -- plastic shrinkage cracking (Newlon 1970) or unstable conditions associated with reactive aggregates (Newlon and Sherwood 1962).

Spalling

Surface spalls are depressions resulting from the separation of a portion of the surface by excessive internal pressure resulting from a combination of forces. As seen in Figure C-5, spalling exposes reinforcement, decreases deck thickness and subjects the thinned section to impact. "Joint spall" is used to designate spalls adjacent to various types of joints, such as that in Figure C-6. The incidence of surface spalling varies considerably within the United States (BPR -- PCA 1969) but where it occurs it is a very serious and troublesome problem. It is related to the use of deicing chemicals, corrosion of reinforcement, traffic volume, and quantity and quality of concrete cover.

Scaling

Scaling is the loss of surface mortar usually associated with the use of deicer chemicals. The severity of scaling is normally expressed qualitatively by terms such as light, medium, heavy or severe. An example of heavy scaling is shown in Figure C-7. The gradual loss of surface by abrasion is sometimes difficult to distinguish from light or medium scaling. Scaling can be locally severe, but generally is not a serious problem if accepted concreting practices are followed.

Popouts

Certain aggregates undergo descriptive expansion during freezing and dislodge portions of mortar to form depressions designated "popouts". An example is shown in Figure C-8. These are usually more of an aesthetic nuisance than a structural problem, but should be avoided, if possible.

Figure C-5(a). Surface spalling.

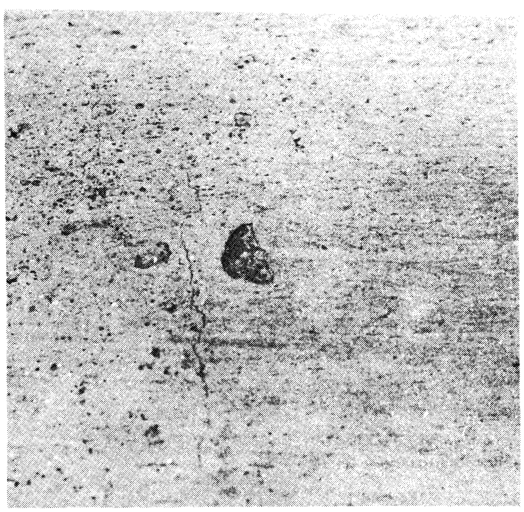


Figure C-5(b) Surface spalling at an early stage of development. Note cracking at location of steel.

Figure C-5(c). Surface spalling at a more advanced stage than C-5(b).

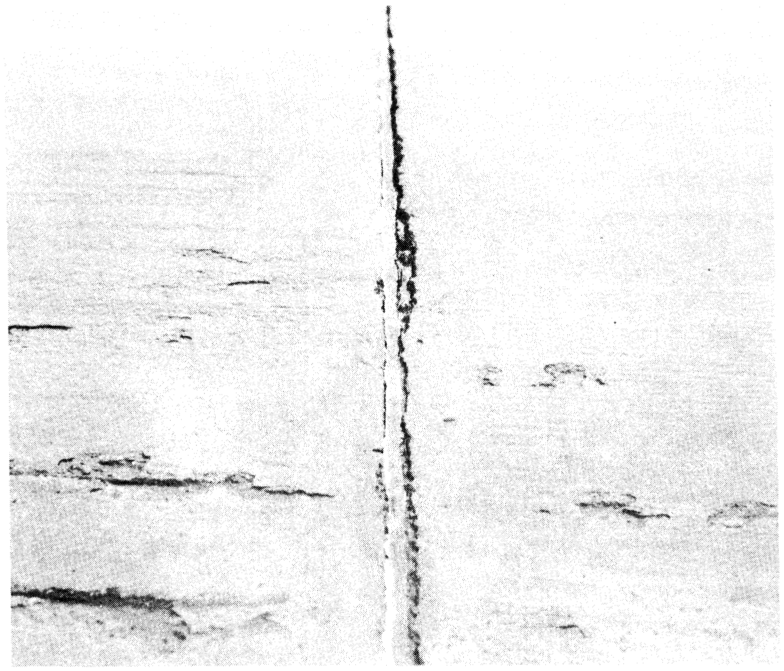


Figure C-6. A small joint spall. Surface spalls associated with the reinforcement are also evident.

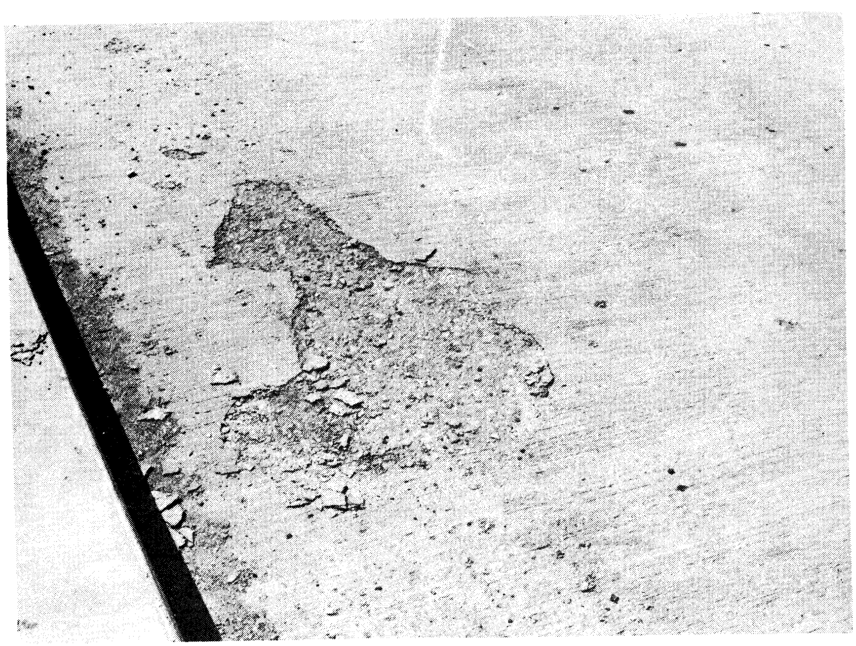


Figure C-7. Surface scaling.

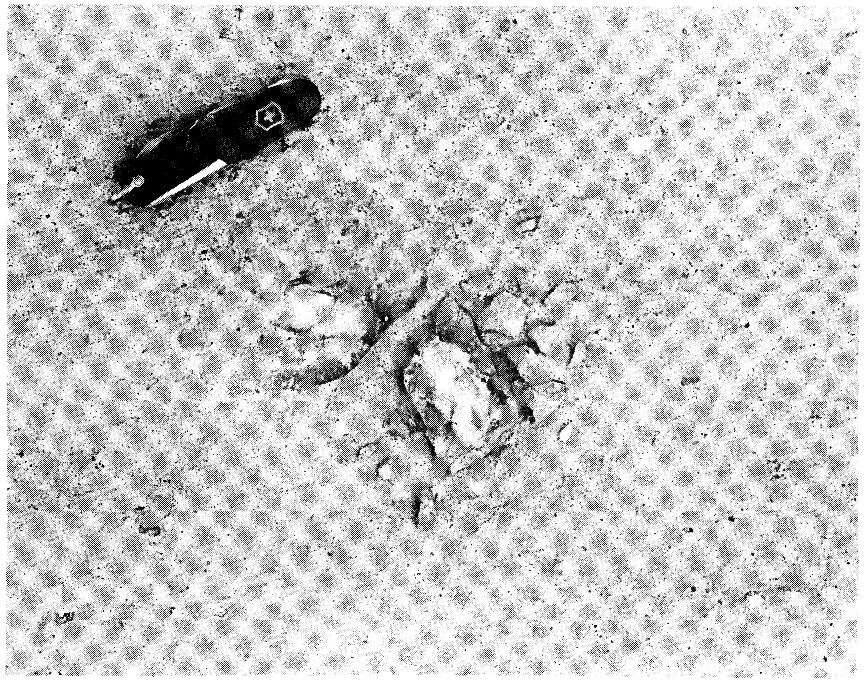


Figure C-8. Typical popout.

Slipperiness

Skid resistance is conventionally expressed in terms of a coefficient of friction determined on wet but ice-free pavements from measurements during panic stops of vehicles, or specially designed measuring trailers. These trailers (ASTM Method E274) permit measurements in shorter lengths than vehicle "stopping-distance" tests and thus can be applied to individual bridge slabs.

The coefficient of friction of the bridge deck surface should not differ substantially from that of the pavement segments that it connects and should have and retain minimum values established for pavement surfaces. Published data for bridge decks are meager but those available for pavements indicate that low skid resistance, or slipperiness, can be influenced by materials and construction practices and subsequently applied coatings. An example of polished aggregate showing low skid resistance is shown in Figure C-9.

2730

Figure C-9. Polished coarse aggregate contributes to low skid resistance.

Summary

Roughness, cracking, spalling, scaling, popouts, and slipperiness are the major defects which result when sufficient attention is not given to the many details which influence their occurrence. Recognition of the interaction of design, materials, and construction practices as well as environmental factors is the important first step toward smooth and durable decks.

APPENDIX D

DEFINITIONS AND FORM USED IN BRIDGE SURVEY

The following criteria were used in the surveys and are taken from PCA — BPR Report No. 5 (1969).

The observations were reported on a standard data sheet as shown in Figure D-1. One sheet was required for each bridge.

Any observed defects were reported for each individual span.

On the data sheets, scaling was reported as an estimated percentage of the affected span's deck area 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 was also placed in the box that designated the most severe scaling condition observed in the span. For example, in Figure D-1, 15 percent of the area of span 2 had an average scaling condition classified as light scale, and medium 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 — were reported as being light, medium, or heavy (L, M, or H). Light cracking meant widely spaced, fine cracks or only a few cracks in the span. Heavy cracking meant closely spaced, wide (prominent) cracks, or many cracks in a span. For example, in Figure D-1, light transverse, longitudinal and map cracking were observed in each span of the bridge, whereas there was no visible diagonal (box 3) or "D" (box 5) cracking in any spans. Light random cracking (box 6) was observed on span 1.

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

Surface spalls were reported as small (box 1) or large (box 2). The number of spalls in each affected span were reported. For example, in Figure D-1, 5 small and 4 large surface spalls were found in span 2.

Joint spalls were reported by the estimated linear footage spalled along the joint. The spalls were classified according to the type of joint on which they occurred: 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 were reported as being few (F) or many (M) in the judgment of the inspector.

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R-300 (6/70) EXHIBIT I

DATA SHEET FOR RANDOM BRIDGE SURVEY INSPECTION REPORT

State Va. County ______ Highway Nos. ______ Bridge No. 4

Year Built 1963 Deck: Uncovered ______ Covered ______ Type of Cover ______

Is detailed construction data available? ______

What type of deck repair or reconstruction has been done? ______

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

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Classification of Deck Deterioration

Span No.		1	2	3	4	5	6	Remarks
Length-ft. Girder Type		56'	64'	64'	56'			
Scaling	1	15 X	15	0	0			1969 ADTC 1,575
	2		X					
	3							
	4							
Cracking	1	L	L	L	L			
	2	L	L	L	L			
	3							
	4	L	L	L	L			
	5							
	6	L						
Rusting	1							
Surface Spall	1	10	5	5	6			
	2		4	5	2			
Joint Spall	1							
	2							
	3							
Popout	1							

Comments:

Date of Inspection 8-6-70 Inspector MN & JD District Office ______

Figure D-1