A SURVEY TO DETERMINE THE IMPACT OF CHANGES IN SPECIFICATIONS AND CONSTRUCTION PRACTICES ON THE PERFORMANCE OF CONCRETE IN BRIDGE DECKS

by

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SUMMARY

In response to its own research and observations in the early 1960's, the Virginia Department of Highways mounted an intensive and extensive effort to improve the performance of concrete in bridge decks. Major elements of this effort included (1) a training and certification program for Department and industry personnel and (2) improved and upgraded specifications for both materials and construction practices.

In 1972 a survey was made of 129 randomly selected bridges constructed after 1966, when all the improvements had been formally instituted. The performance of these bridges was compared with that of a similar sample that had been surveyed in 1961. In addition to the visual observation of performance measurements of electrical corrosion potentials and depth of concrete cover were made in the 1972 survey.

Based upon this survey the following conclusions and recommendations were drawn:

(1) The frequency of early bridge deck scaling has been dramatically reduced by the upgrading of specification requirements and construction practices. Several specific changes such as increased air contents, use of linseed oil treatments as well as increased awareness of the problem all contribute to this improvement. Because concrete susceptible to scaling usually exhibits the defect at an early age this is an encouraging result. The elimination of scaling was a major target of the specification upgrading effort. The success of this effort is evident.

(2) Transverse and random cracking are indicated to be more frequent than before the upgrading. The reason for the increase in transverse cracking is not apparent and there is other evidence that the indicated increase in random cracking is related to closer observation and differences in classifications rather than to real causes. The severity of cracking does not seem serious enough to warrant attention. Real differences, if any, will become more apparent with time.

(3) The frequency of all other defects is very low. Based upon previous studies this will undoubtedly increase with age, traffic, etc., but experience suggests that serious problems are indicated at comparatively early ages.

(4) The measured average cover over reinforcement is fortunately significantly greater than that specified. For the two levels of cover specified, 8 and 16 percent of the measurements are less than required. This is believed to reflect an acceptable level of control.

(5) Ninety-five percent of the spans have average corrosion potentials below 0.20 volt, which indicates no active corrosion. On one percent of the spans the average values are above 0.40 volt, which suggests the presence of active corrosion. The potential for corrosion will increase with age and exposure to deicing chemicals.

(6) The techniques developed for the BPR-PCA survey in 1961 and used in previous studies by the Research Council provide reproducible and useful evaluations of performance based upon visual observations. The procedures reflect general trends and levels as opposed to detailed causes and effects.

(7) When the bridges to be surveyed are similar in age and condition and when the sample is sufficiently large, observations on a single randomly selected span provide the same results as observations of all spans on the bridge. Stated in other terms, the observations of spans rather than bridges appears to be a valid approach.

RECOMMENDATIONS

(1) Because the level of the performance indicated has improved with respect to the deficiencies which were the objectives of the upgrading to current specification and construction practices, and because the remaining defects continue to be infrequent in occurrence, the procedures for control and acceptance of bridge deck concrete now in use should be continued.

(2) A resurvey of the bridges should be scheduled in 1977-78. The decks will then be five to ten years old.

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Coincident with the increasing national and local concerns over the premature deterioration of concrete in bridge decks, the Virginia Department of Highways in the early 1960's mounted a research effort, instituted modifications of its specifications, and initiated extensive specialized training programs with the objectives of extending the service life of these decks. The Research Council in 1963 began studies on construction practices, particularly finishing methods, which were described in two reports (Davis, North and Newlon 1971, Newlon 1972). Although not directly a part of the research effort, Virginia was one of eight states included in the comprehensive nationwide study of bridge deck performance conducted by the Portland Cement Association and the Bureau of Public Roads (BPR-PCA 1969).

The data from the BPR-PCA Survey were voluminous and some of the important results are summarized in Table 1.

Table 1

Frequency of Occurrence of the Most
Commonly Observed Defects in the BPR-PCA Study (1969)

(values given as a percentage of the total spans surveyed within a state)

Defect	7 States	Virginia
Cracking (all types)	69.7	33.2
Spalling	8.1	0.4
Scaling	22.9	43.9

The bridges surveyed in Virginia ranged in age from 1 to 21 years at the time of inspection.

As compared with those in the other states, the bridges in Virginia showed less cracking, substantially less spalling, and significantly more scaling. The lower incidence of cracking was attributed to a greater proportion of simple spans in the Virginia sample. It was also concluded that the higher frequency of scaling was due to the comparatively late adoption of air entrainment by Virginia.

Based upon the initial results from these research studies and its own experiences, the Virginia Department of Highways initiated several significant operational changes. While these were intended to upgrade performance in general, special attention was directed toward factors associated with scaling, which was recognized at that time to be very prevalent.

In January 1963, a program was instituted for certification for contractor and Departmental personnel involved with the production of concrete. This program included classroom and field instruction and testing. Beginning on contracts advertised after September 1963, no concrete could be delivered to highway projects unless there was a certified person at the producing plant. Inspector awareness and competence were increased through special schools, the certification program, and continual emphasis upon the factors important to the production of high quality concrete. Early in the Council's research study the preliminary findings were presented in instructional sessions held in 1964 in each construction district and attended by about 300 operating personnel. In 1966 the specification requirements were substantially upgraded based upon recommendations from the BPR-PCA and Council studies.

Particular emphasis was placed upon insuring a high level of air entrainment, which was recognized in the Council's study (Newlon 1971) to be the most important factor in providing resistance to deicing chemicals. Also in 1966 the use of linseed oil treatments on bridge superstructures was made mandatory. The factors leading to this decision as well as subsequent evaluations have been previously reported (Newlon 1970). The progressive changes in specification requirements for bridge deck concrete between 1938 and 1970 are shown in Table 2. (Note: In 1973 the minimum cement content was reduced to 6 3/4sk/cy (376 kg per cu m) as it had been during the period 1966-1970).

The extent to which these efforts either singly or in combination have improved the performance of concrete in bridge decks is difficult to assess quantitatively. The degree to which the "bridge deck problem" continues for decks built under the more stringent requirements is also a matter about which there is some controversy. It was, therefore, deemed appropriate to evaluate the performance of decks built under the upgraded requirements and procedures for comparison with observations from the earlier study. (Davis, North and Newlon 1971).

OBJECTIVES

As stated in the work plan (Newlon and Smith 1972) the objectives were:

(1) To assess the condition of a randomly selected group of bridges designed and constructed since 1966 under Virginia's upgraded deck specifications as compared with the performance of a similar group of bridges constructed under former specifications and surveyed in 1961.

(2) To assess the effectiveness of the newer and more stringent specifications as a deterrent to various forms of deck deterioration.

(3) To obtain base data from comparatively new bridges for comparison in future surveys.

Table 2

Requirements for Bridge Deck Concrete 1938-1970

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

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

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

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

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

PROCEDURES

The BPR-PCA method (BPR-PCA 1969) of inspection was utilized to evaluate by visual inspection the nature and extent of defects. This method had been previously used with good results in a study of some decks in Virginia (Davis, North and Newlon 1971). The details and rationale for the method have been previously published as well as the usefulness of the results.

Briefly stated, the decks were observed by a team of inspectors using a clearly defined classification system. The forms of deterioration—classified as to type, extent, and severity—were scaling, spalling, cracking, rusting, and popouts. In addition to the visual survey, two other characteristics were determined: (1) the depth of cover over the uppermost reinforcing steel as measured by a pachometer, and (2) the electrical half-cell potential determined in accordance with a method initially developed by Stratfull (Tremper, Beaton, and Stratfull 1957) as a possible indication of future corrosion of reinforcement. These two characteristics were included because of their potential importance to corrosion of reinforcement and spalling as described in a supplemental work plan for the project (Newlon 1972).

A copy of the survey form and definitions utilized in the visual survey are given in Appendix A. One member of the inspection team had also participated in a 1970 resurvey of the 1961 survey sample.

Electrical half-cell potential measurments were made on each deck at the intersections of a five foot (1.5m) grid. The procedures used and equipment employed were those described by FHWA Region 15. These are contained in a report of the demonstration in Virginia (FHWA-15 1971). These procedures were in wide use at the time and are described in Appendix B. Although some variations and inconsistencies were anticipated because the method did not differentiate among ways of connecting to reinforcement, moisture content of concrete, etc., it was used as described in Appendix B. Refinement of the procedure was beyond the scope of this project. Subsequent work, particularly that by Clear and Hay (1973), has identified several problems which may influence significantly the quantitative results obtained with the method and interpretation of these results as indicating corrosion or no corrosion. A major constraint on the results obtained was the fact that the ground connection was not made directly to the reinforcement but was made indirectly through exposed metal, utilizing the fortuitous connections of this exposed metal to the reinforcement. Ground connections were made giving preference in decreasing order to (1) steel beams, (2) bolts in handrails, (3) joint cover plates, and (4) metal connections. In all cases, the ground locations were recorded and marked with a chisel for use in future surveys. Thus, there is an unknown element of variation which will affect particularly comparisons of potential measurements among the various decks. The uncertainty would be expected to be less within the same span, spans on the same bridge or in the event that future measurements are made with ground connections at the same locations.

No recognition was taken of transient high potential readings such as described by Clear and Hay (1973). Consistent readings were usually obtained over the surface of a given span so that errors from short-term effects are probably not significant in this project.

Depths of concrete cover over the uppermost reinforcement were measured at 30 of the grid intersection points on each deck, using a James Pachometer Model C4946. While the corrosion potential measurements required a considerable expenditure of effort, they were a subordinate part of the study whose principal objective was the evaluation of performance based upon visual observations.

BRIDGES SURVEYED

In order to provide a comparison with the results from the earlier surveys made in 1961 and 1970, a sampling of bridges from the five year period 1968-1972 was selected for observation and comparison with the group of bridges built during the years 1957-1961 and inspected in 1961. During the period 1968-1972 approximately 755 bridges were constructed under the upgraded specifications adopted in 1966. These bridges contained approximately 2,500 spans. Using the relationship developed for the BPR-PCA survey (BPR-PCA 1969) the sample size was selected by the following relationship:

$$n = \frac{1}{.0064 + 1/N}$$

where n = sample size

N = number of bridges available for study

Based upon this formula 130 bridges were randomly selected for the visual survey. During the survey one was inadvertently missed so that 129 were actually inspected. These bridges contained 436 spans. A listing of the bridges is contained in Appendix C. The half-cell potentials and cover depths were determined after the visual survey. In 24 cases, because of the design of the bridge or because of difficulties in obtaining a usable ground connection, the depth and potential measurements were not made. Thus, visual observations, cover depth and corrosion potential measurements were made on 105 bridges (341 spans) while visual observations only were made on 24 bridges (59 spans).

Because a major goal of this study was to determine the performance of a comparable set of bridges in 1961 and 1972, the distribution by age within the five-year period is important and is shown in Table 3.

Table 3

Distribution by Age of the Bridges Surveyed in 1961 and 1972

Age at time of Survey, yr.	Yr. Built	1961 Survey Number	% of Total	Yr. Built	1972 Survey Number	% of Total
4-5	1957	7	18.4	1968	18	14.0
3-4	1958	11	28.9	1969	38	29.5
2-3	1959	7	18.4	1970	41	31.8
1-2	1960	10	26.3	1971	25	19.4
0-1	1961	3	7.8	1972	7	5.4
		38			129	

While there is some difference, primarily between decks that were two or three years old, the samples are roughly comparable, particularly in the case of the extreme situations; i.e., the oldest and youngest decks. Thus, comparisons of performance drawn from the two samples should be valid.

The bridge types are identified according to (1) material comprising the beams, (2) type of member, and (3) whether continuous or simple spans and composite or noncomposite. The designations used for group (1) are: structural steel (SS), prestressed concrete (PS), and reinforced concrete (RC); for group (2): box girder (BG), deck girder (DG), I-beam (IB), solid slab (SS), truss (TA); for group (3): the first letter designates simple (S), or continuous (C), while the second indicates composite (C) or noncomposite (N). These designations are included for each bridge in Appendix C.

Each bridge on the listing of all bridges constructed during the period 1968-72 was given a number which was used in the random selection. This "random number" was the number used to identify the bridge in all Council records. This number, along with other identifying information, is also included in Appendix C.

RESULTS

Visual Survey

The results warrant attention from two perspectives. The frequency of occurrence of the specific defects is of importance as an indicator of performance as is the change in frequency between the two surveys, which would indicate the influence of upgraded practices. The sampling plan was developed using statistical techniques to provide a 95 percent probability that the results from the samples would be within ± 8 percentage points of the actual value. No statistical test for significance of the differences reflected in the data was made but the value of ±8 percentage points might be used as an indication of meaningful differences.

In several cases the difference between frequency of occurrence observed in the two surveys is probably significant, but the frequency level is so low that the defect itself is not of concern.

The results from the 436 spans are summarized in Table 4 in the same format as that used in the BPR-PCA report (1969) and the Council's earlier report (Davis, North, and Newlon 1971). Study of the results in Table 4 indicates two obviously significant differences. The incidence of scaling is significantly less in the 1972 sample than was the case for the comparable sample in the 1961 survey. Scaling on bridges at early ages has for practical purpose been eliminated. At the same time, cracking is more prevalent (twice as frequent) in the 1972 group than in the comparable group surveyed in 1961. Attemps to relate the increase in transverse cracking to causative factors were unsucessful. In the 1972 sample transverse cracking increased in the expected manner with age, length of span and traffic volume. No increased transvere cracking was found on continuous spans as compared with simple spans. Based upon the resurvey of the 1961 sample made in 1970, there is evidence that random cracking was vastly underestimated in the 1961 survey. It is thus probable that the indication of increased random cracking reflects a difference in inspector judgment rather than an actual increase in cracking. The other defects are essentially the same in both samples and in both cases are not frequent in occurrence.

Table 4

Occurrence of Defects in Spans

	1961 Survey		1972 Survey	
Span Defects	Number	Percentage	Number	Percentage
No Scaling	90	70	426	98
Scaling	40	30	10	2
No Cracking	105	80	260	60
Cracking	25	20	176	40
Transverse	15	12	126	29
Longitudinal	4	3	5	1
Diagonal	2	1	2	0.5
Pattern	1	1	12	3
"D"	0	0	0	0
Random	9	7	93	27
No Rusting	130	100	426	98
Rusting	0	0	10	2
No Surface Spalling	129	99	436	100
Surface Spalling	1	1	0	0
No Joint Spalling	130	100	434	99.5
Joint Spalling	0	0	2	0.5
No Popouts	124	97	434	99.5
Popouts	4	3	2	0.5

It is of particular interest to note that only two spans showed spalling and in neither survey did the spalled spans make up 1 percent of the total. As indicated in the previous report (Davis, North, and Newlon 1971) when surveyed in 1970, spalling on the decks in the 1961 sample had increased to 10 percent but still was considerably less prevalent than on decks in many of the other states included in the random survey at much earlier ages. Although the frequency of spalling is not so great nor does it occur so early as some of the other defects, particularly scaling and cracking, where it does occur it is a very troublesome and expensive defect to correct. It thereby merits continued attention.

The results shown in Table 4 reflect the presence of the various defects regardless of severity. Because the entire sample was from relatively new bridge decks on which the defects were infrequent and comparatively minor, in Table 5 the data are presented in a form which combines the spans showing light scaling and transverse cracking with those showing no defects. These were the two most prevalent defects. A similar technique was also used in the earlier report (Davis, North, and Newlon 1971) with the belief that it minimizes the differences attributable to judgements of the individuals conducting the two surveys.

Table 5

Occurrence of More Severe Scaling and Transverse Cracking on Spans

Span Defect	1961 Sample %	1972 Sample %
No or Light Scaling	96	99.5
Medium, Heavy, Severe Scaling	4	0.5
No or Light Transverse Cracking	93	97
Medium or Heavy Transverse Cracking	7	3

The results in Table 5 would indicate that the increase of cracking and the decrease in scaling are predominantly in the light category. Visual comparisons of the data are presented in Figures 1-6. From these figures comparisons can be made among the characteristics of all decks surveyed in 1961, some of which were as much as 20 years old, as well as the portion built during the five-year periods immediately preceding the two surveys. These latter portions would be the basis for estimating the influence of specification changes and other efforts to improve performance.

The significant reduction in the frequency of scaling is evident in Figure 1. As shown in Figures 2 and 3, there is no relationship in the 1972 sample between scaling and either age or traffic volume. This might suggest that significant scaling will not develop since concrete susceptible to scaling usually exhibits symptons early in its exposure to deicers. Such exposure is more common on the bridges with higher traffic volumes than on more lightly traveled roads. The reduction in scaling is encouraging because this was the defect which was indictated in the initial survey to warrant major attention, as evident from Table 1.

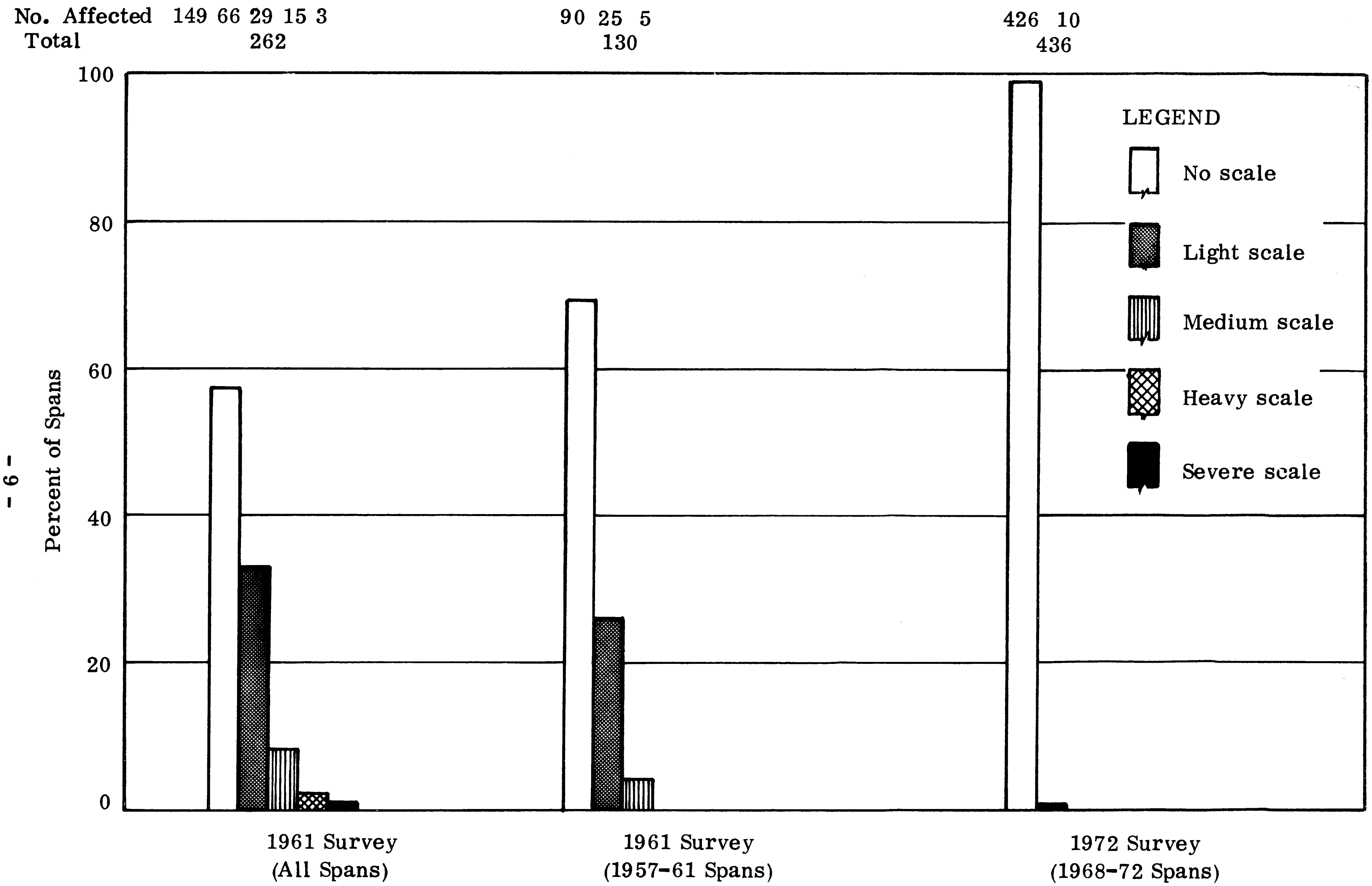
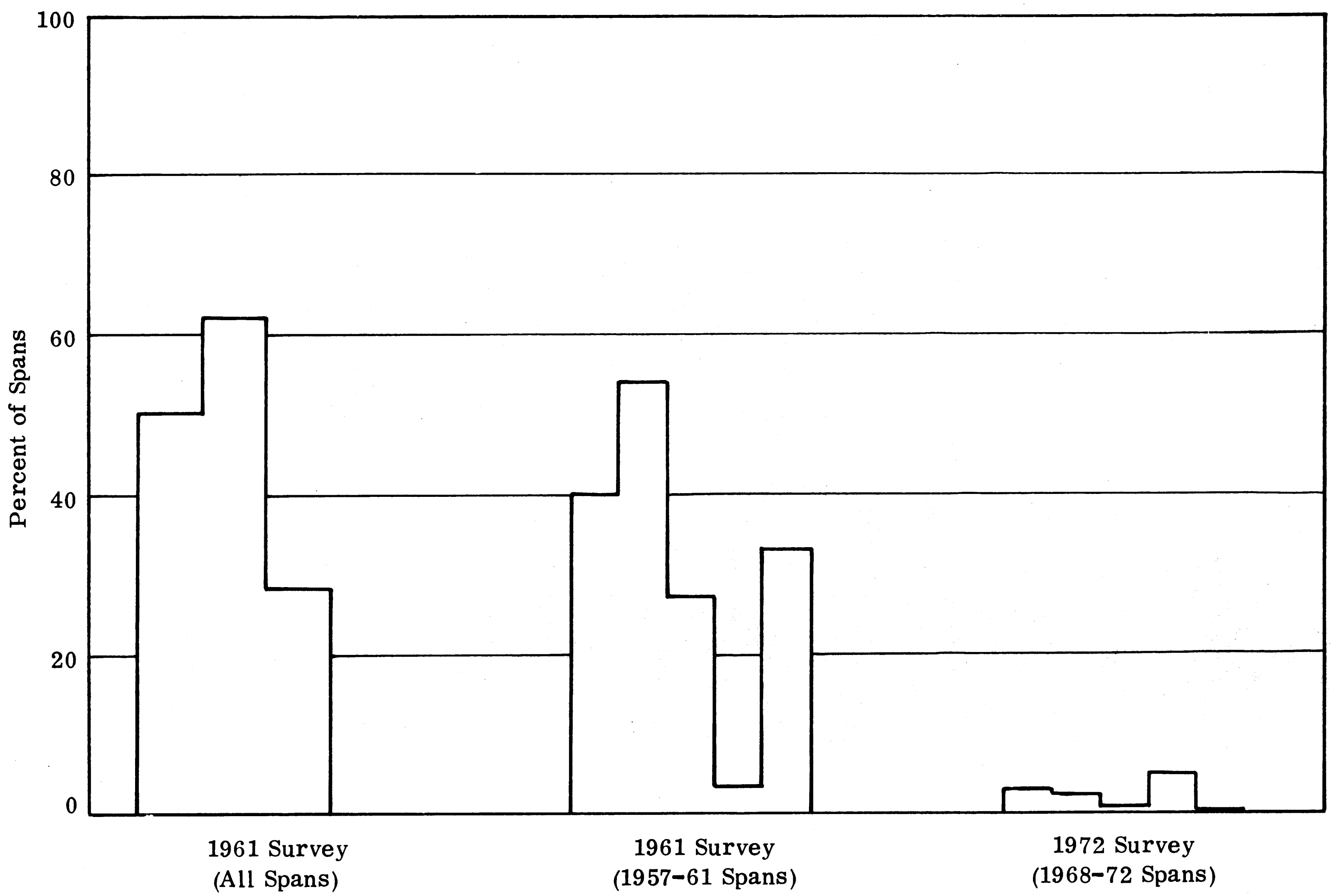


Figure 1. Occurrence of scaling (average scaling condition).

2467

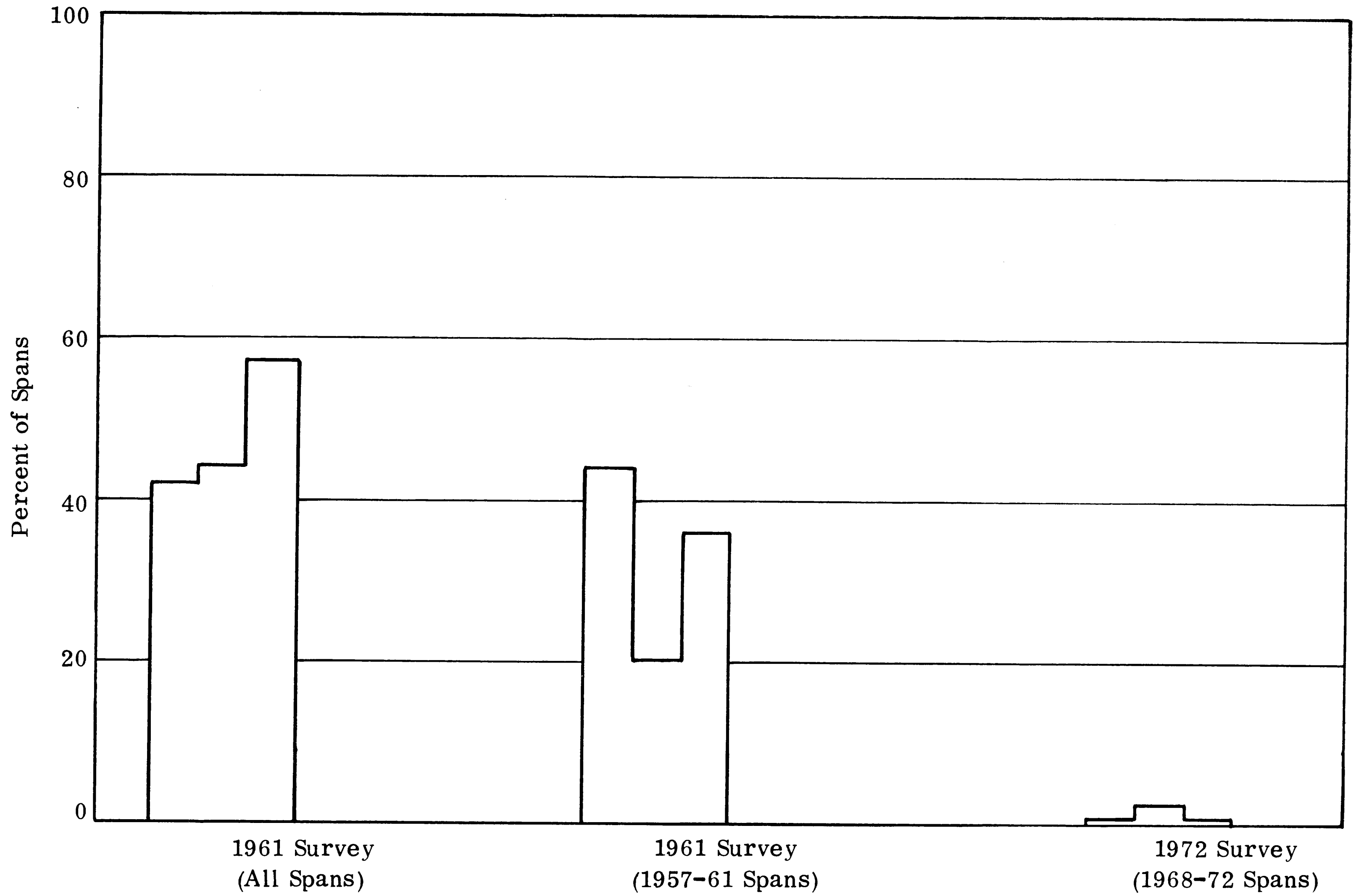
Age *	(1)	(2)	(3)	'57	'58	'59	'60	'61	'68	'69	'70	'71	'72
No. affected	4	70	39	11	16	6	1	6	2	3	1	4	0
Total	8	112	142	28	30	23	31	18	71	128	129	85	23



*(1) 1940–47
(2) 1948–55
(3) 1956–61

Figure 2. Influence of age on occurrence of scaling (all types).

Traffic Volume*	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
No. affected	46	49	18	22	11	6	0	8	2
Total	115	114	33	50	63	17	128	272	36



*(1) ADT 1-750
(2) ADT 751-7500
(3) ADT > 7500

Figure 3. Influence of traffic volume on occurrence of scaling (all types).

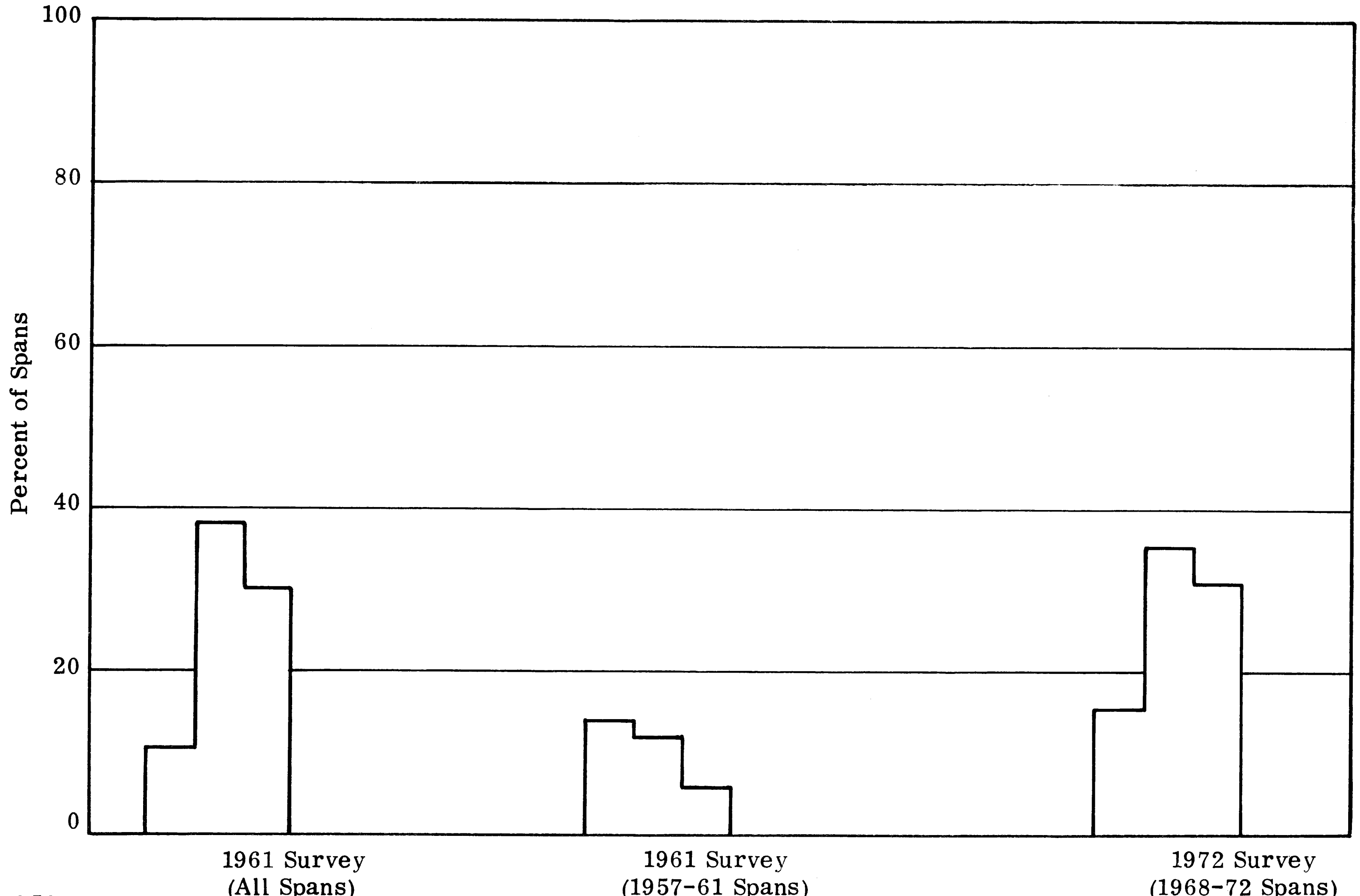
2470

Length *	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
No. affected	13	43	10	9	4	1	14	84	28
Total	115	114	33	67	59	4	118	270	48

* (1) < 45 ft.
(2) 45 ft.-90 ft.
(3) > 90 ft.

Figure 4. Influence of span length on occurrence of transverse cracking.

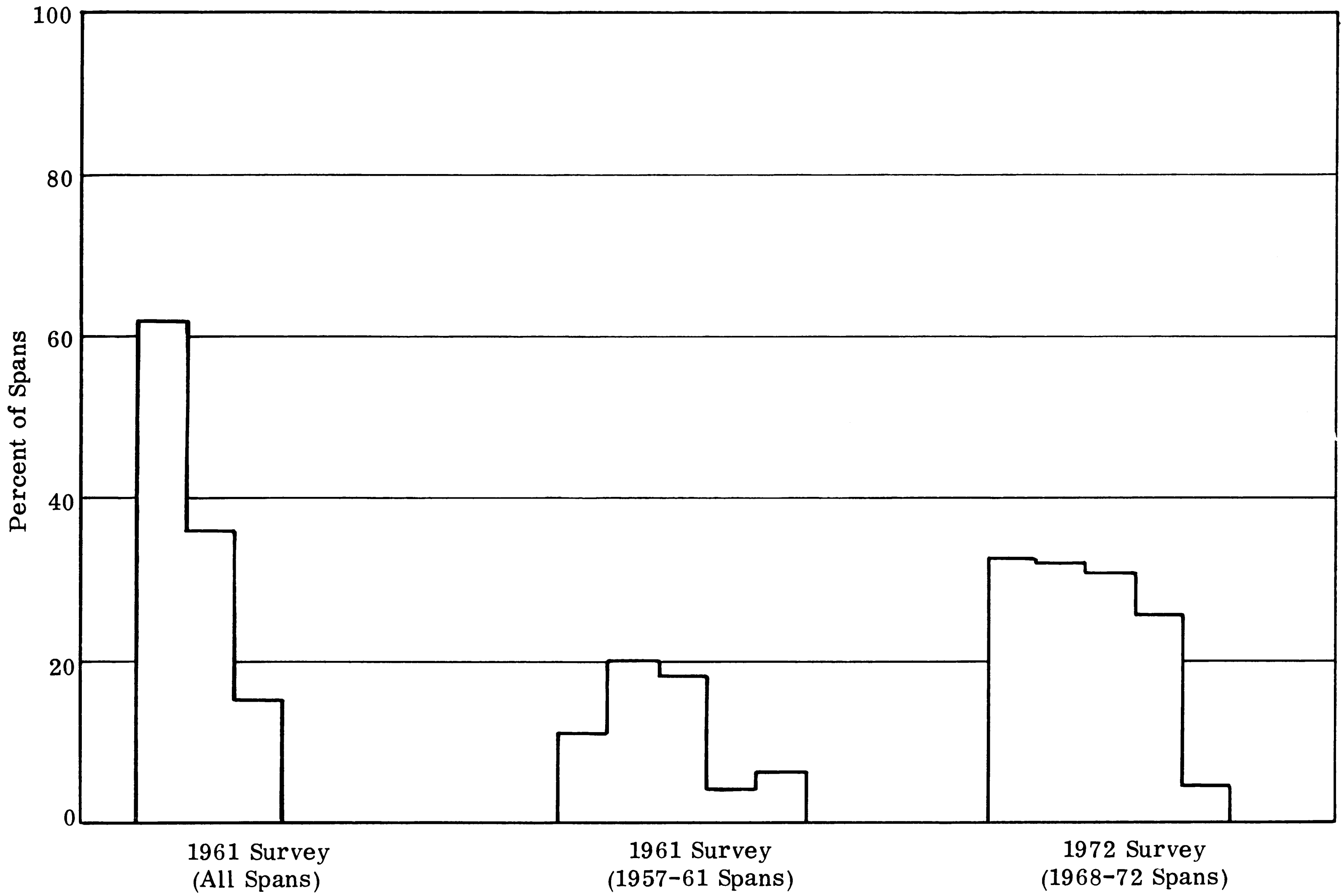
Traffic Volume *	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
No. affected	13	43	10	7	7	9	19	96	11
Total	115	114	33	50	63	17	128	272	36



*(1) ADTC 1-750
(2) ADTC 751-7500
(3) ADTC > 7500

Figure 5. Influence of traffic volume on occurrence of transverse cracking.

Age *	(1)	(2)	(3)	'57	'58	'59	'60	'61	'68	'69	'70	'71	'72
No. affected	5	40	21	3	6	4	1	1	23	41	38	22	1
Total	8	112	142	28	30	23	31	18	71	128	129	85	23



* (1) 1940-47
(2) 1948-55
(3) 1956-61

Figure 6. Influence of age on occurrence of transverse cracking.

The data in Figures 4 through 6 reflect the influence of span length, traffic volume, and age on the frequency of transverse cracking. The relationships for the 1972 sample are those that would be expected in that the frequency of transverse cracking increases with span length, traffic volume, and age.

A comparison was also made of the influence of continuity on cracking frequency. Only ten of the 129 bridges were designed for continuity. Of these ten bridges three were solid concrete slabs and seven were supported by structural steel. Of the seven steel beam bridges, three contained transverse cracking as did one of the three concrete slab spans. The frequency of cracking on the continuous spans was about the same as on all spans. There also appears to be no difference in the severity of cracking between the two types. The remaining defects are of no consequence at this time.

As noted previously the sample size was determined on the basis of bridges rather than spans. For each of the 129 bridges selected, all of the spans were inspected. An analysis was also made to compare the results based upon data from all 436 spans with those from an analysis using data from one randomly selected span from each of the bridges. The comparison is given in Table 6.

Table 6

Defects in Percent as Indicated by the Total Sample and a Single Span from Each Bridge

Defect	436 Spans	129 Spans
No Scaling	98	99
Scaling	2	1
No Cracking	60	59
Cracking	40	41
Transverse	29	30
Longitudinal	1	2
Diagonal	0.5	0
Pattern	3	4
"D"	0	0
Random	27	20
No Rusting	98	96
Rusting	2	4
No Surface Spalling	99.5	99
Surface Spalling	0.5	1
No Joint Spalling	100	100
Joint Spalling	0	0
No Popouts	99.5	99
Popouts	0.5	1

The comparison shown in Table 6 indicates the results from observations of a single randomly selected span are the same as when all of the spans are included. This suggests that in future surveys observations of a single randomly selected span will be sufficient to provide valid information on performance characteristics.

Depth of Cover

Pachometer readings were taken at 30 randomly selected grid points using a James Model 4946. These readings were taken to indicate the clear cover over the uppermost steel, normally the transverse reinforcement. Calibrations were made on a series of slabs fabricated in the laboratory with carefully positioned reinforcement prior to initiating field measurements. Checks on these slabs were made at weekly intervals throughout the measuring period. These periodic calibrations always gave depths within ± 1/8 inch (3mm) of the actual values. Recently attention has been drawn to possible errors associated with the Model C4946 (FHWA 1974). No indications of such variations were observed in this study. The instrument also has been shown in extensive evaluations (Weber, et al. 1972) to indicate cover depths within ± 1/8 inch (3mm).

The results from the pachometer determinations are illustrated in Figures 7-9 in the form of histograms. The data are shown in the form of frequency distributions in Figures 10-12. Readings were made at 30 randomly selected grid points of the five-foot grid on 339 spans where corrosion potential measurements presented later were also made. For the 117 bridges tested the total number of measurements was 10,170. A considerable range of indicated cover is evident in Figure 7. The distributions in Figures 7-9 appear to be approximately normal. The average cover from all measurements is 2.40 inches (61mm) with a standard deviation of 0.49 inch (12mm).

The decks surveyed were constructed under specifications that required different amounts of clear cover. The predominant values were *1.69 inches (43 mm) or **1.94 inches (49 mm). The results for decks with these specified covers are shown in Figures 8-9 and 11-12. In both cases, the indicated average cover was well below the minimum specified. For a minimum specified cover of 1.69 inches (43 mm) the measured average was approximately 5/8 inch (16 mm) greater. For the minimum specified cover of 1.94 inches (49mm) the measured average was approximately 1/2 inch (13 mm) greater. The difference between the two averages is 0.16 inch (4mm), or slightly over one-half of the difference between the specified values. As seen in Figures 11 and 12 for the two specified values, 8 and 16 percent of the steel had less than the specified clear cover. This is a very fortuitous situation since the cause of spalling, penetration of chlorides, is greatly dependent upon the amount of cover. The variability, while large, compares closely with the results from other published studies of variability of steel placement in bridge decks and guidelines included in the recently adopted Recommended Practice of the American Concrete Institute (ACI 345-1973). The AIC Practice

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* 1.69"=1 11/16"
** 1.94"=1 15/16"

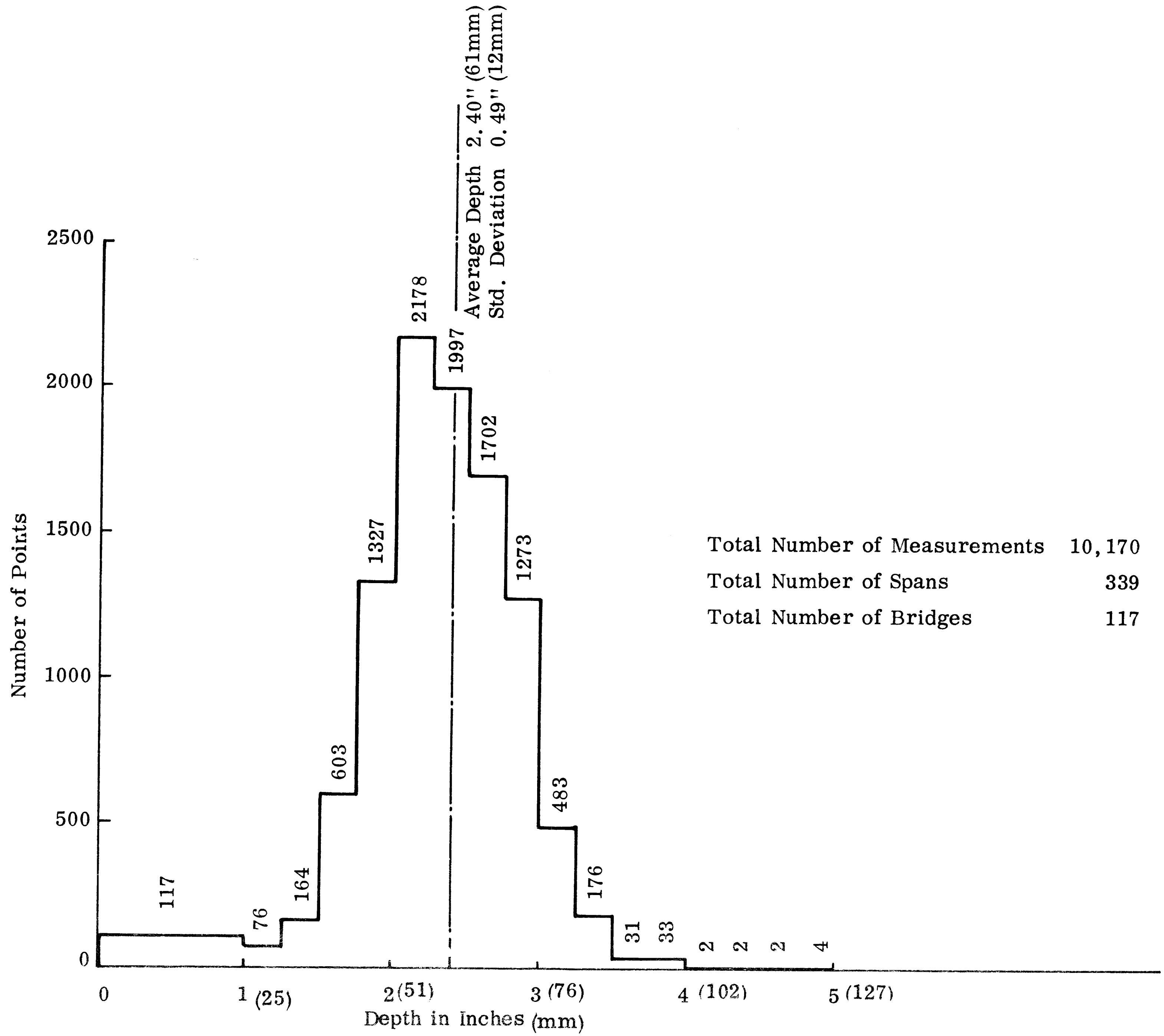

Figure 7. Distribution of measured clear cover for all spans.

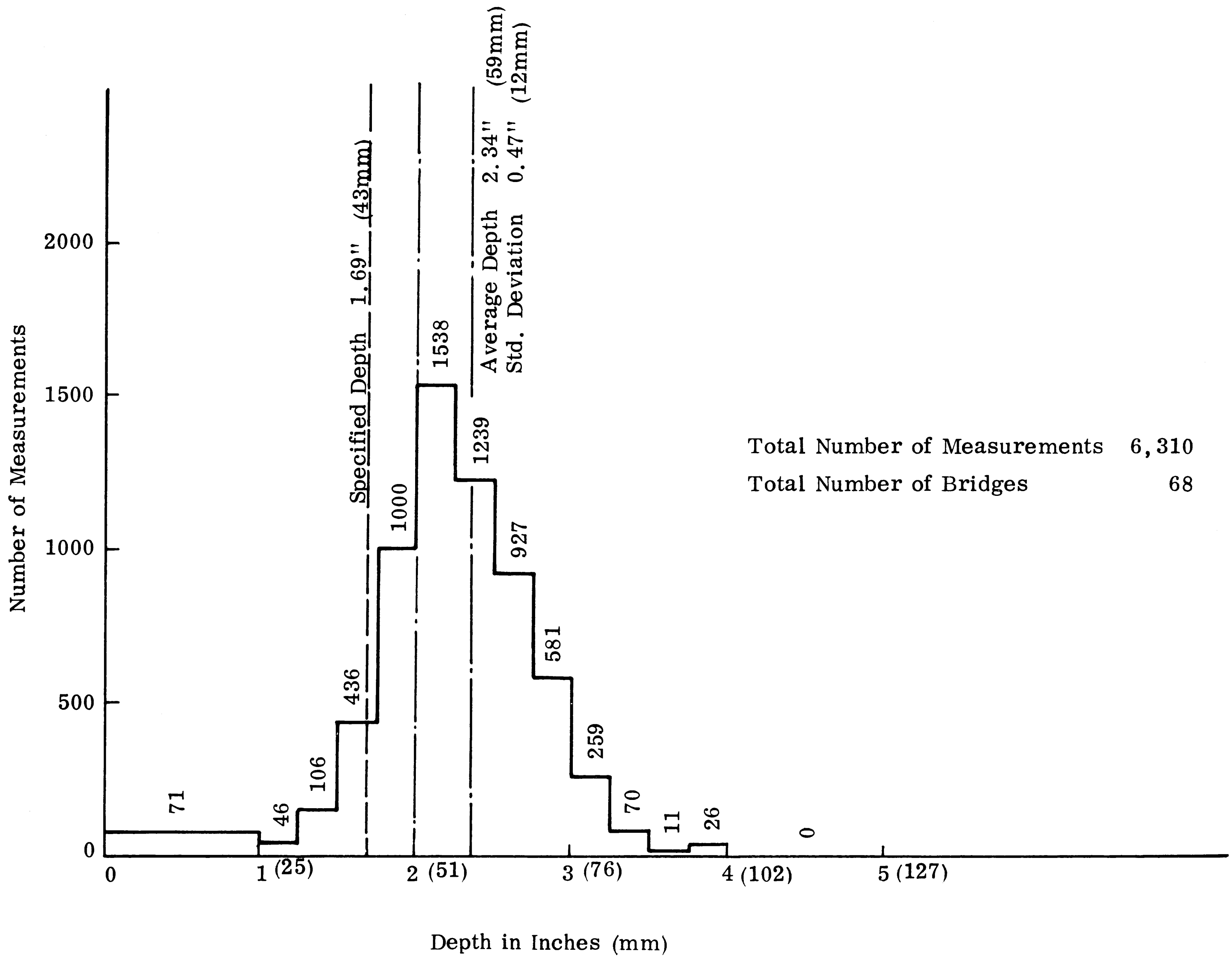

Figure 8. Distribution of measured clear cover for spans with specified clear cover of 1.69" (43mm).

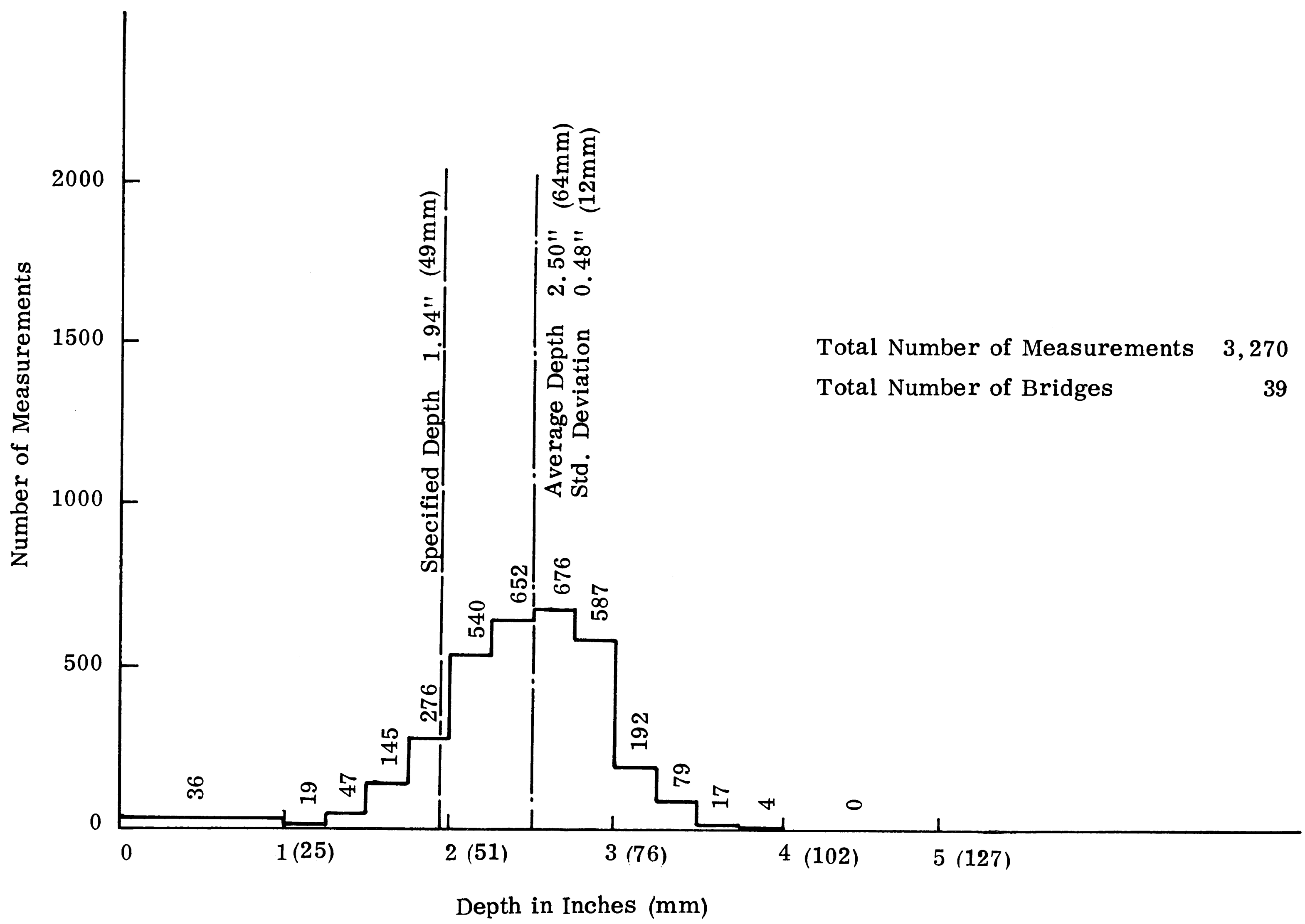

Figure 9. Distribution of measured clear cover for spans with specified clear covers of 1.94" (49 mm).

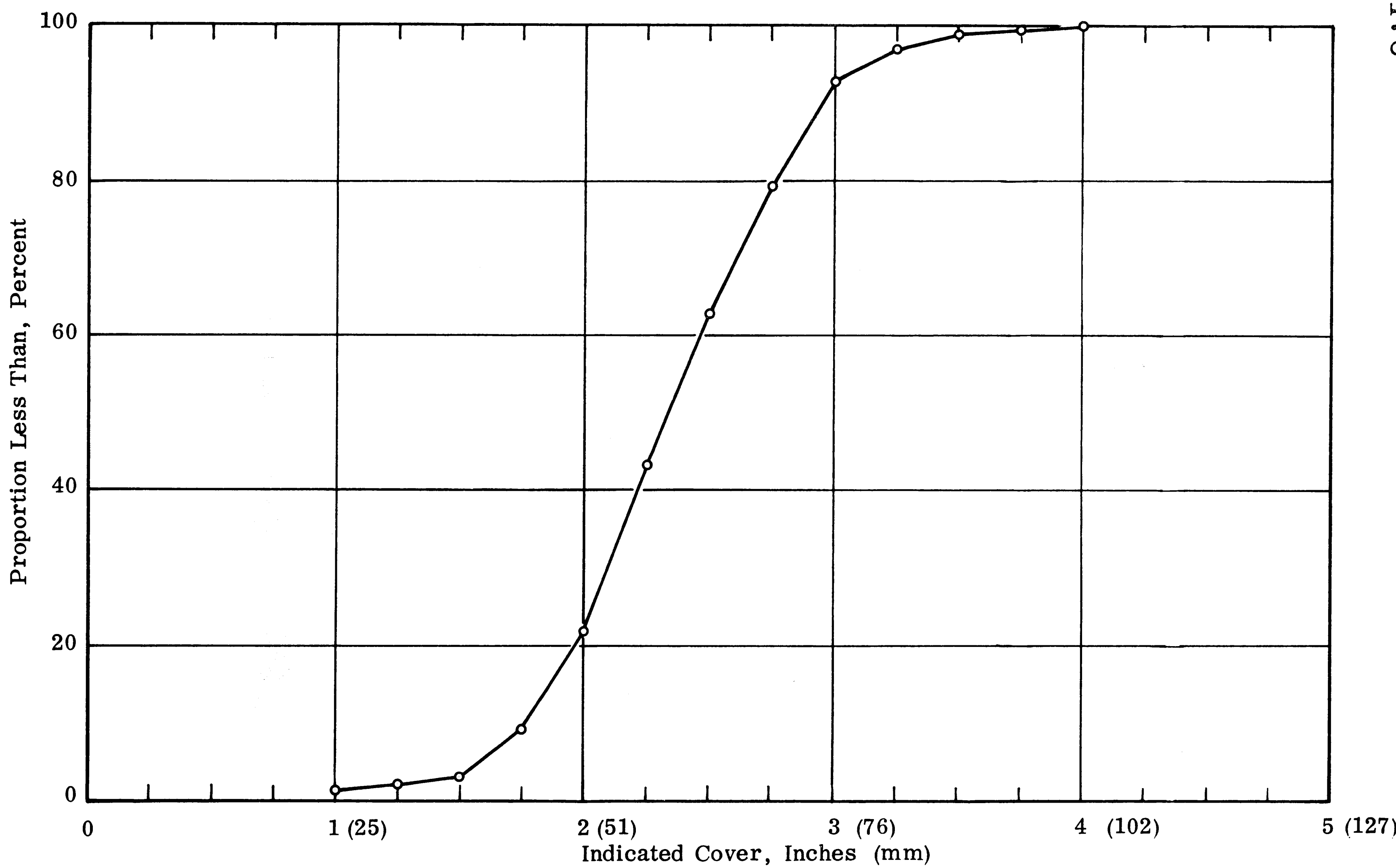

Figure 10. Cumulative frequency of measured clear cover for all decks.

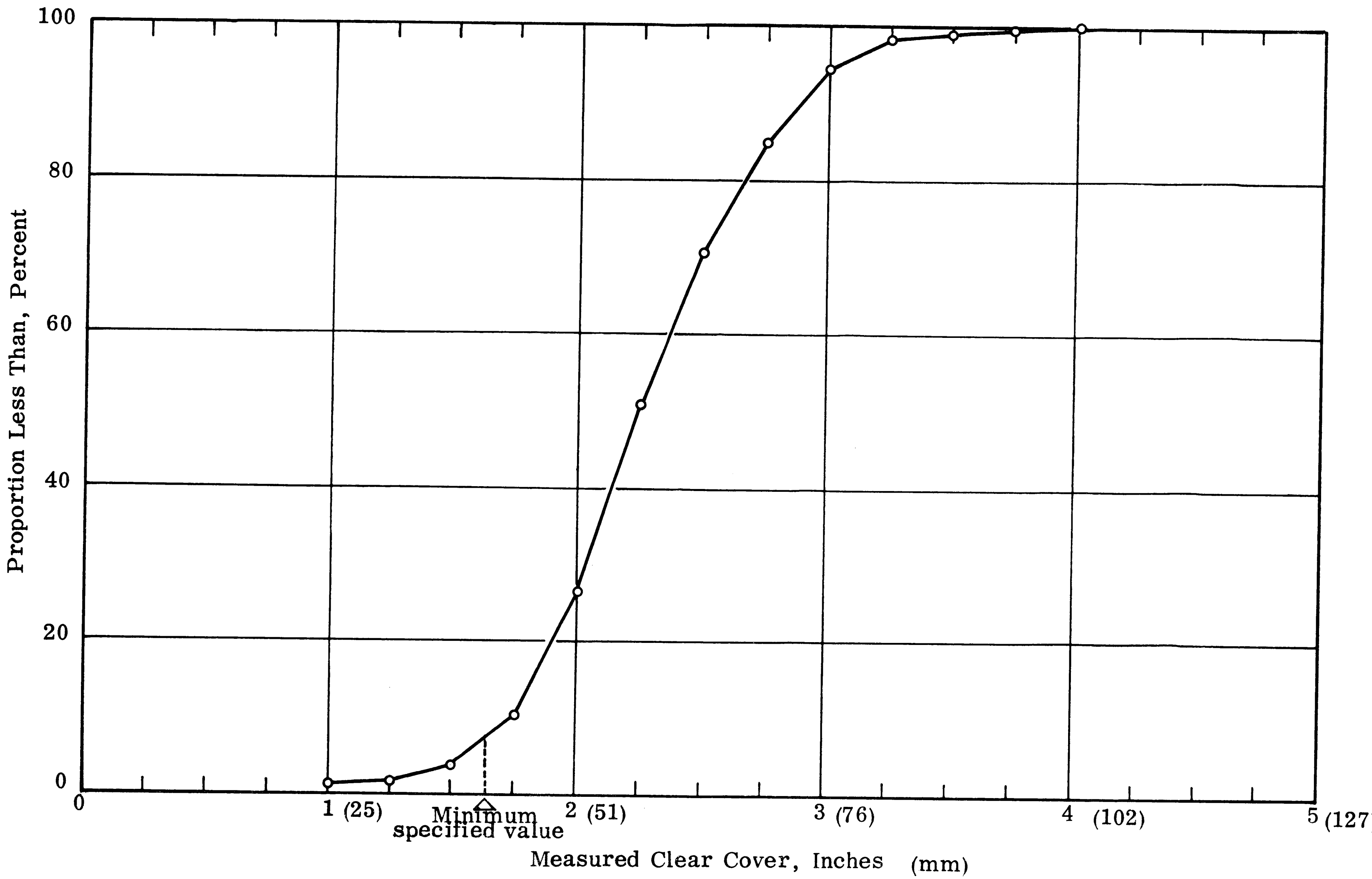

Figure 11. Cumulative frequency of measured clear covers for decks with specified clear cover of 1.69" (43mm).

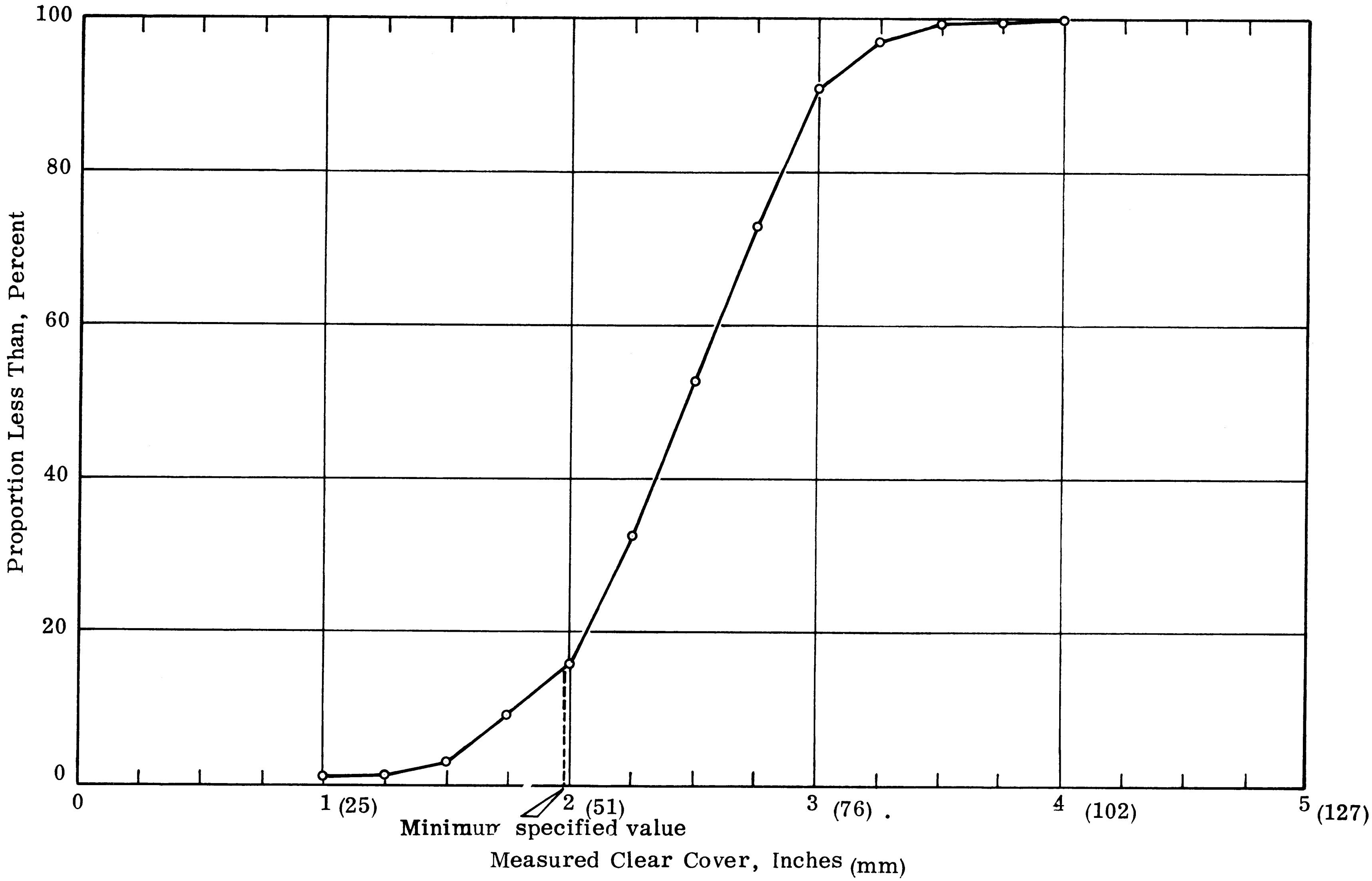

Figure 12. Cumulative frequency of measured clear cover for decks with specified clear covers of 1.94" (49mm).

indicates the need for a tolerance of ± 3/8 inch (10mm) in steel placement. The fact that the average cover is significantly greater than that specified indicates that most contractors have recognized the variability and are making appropriate allowances.

Included in the measurements are instrument errors as well as the fact that in some cases the longitudinal steel was above and in some cases below the transverse or major reinforcement. The fact that the steel is generally at a lower elevation than specified is consistent with a recognition of the obvious influences of construction practices. With the exception of gross errors in establishing elevations, most of the things which happen to reinforcement would tend to lower its final elevation. These include sag between supports, bending under foot traffic, and "racking" or collapse of the supports.

Corrosion Potential

Using the procedures and equipment described in Appendix B, corrosion potential measurements were made on 341 spans at the intersection points of a five-foot grid. The total number of measurements was 34,647.

The corrosion potential measurements are thus voluminous. A typical example of the methods used to reduce the data is given in Figures 13 and 14, for a single span. The potential measurements were first plotted on a grid as illustrated in Figure 13. Equipotential contours were used to delineate the readings within each of the five groupings as follows:

< 0.049
0.050 - 0.199
0.200 - 0.349
0.350 - 0.449
> 0.450

The contour diagram for the span is given in Figure 14, along with the cumulative frequency diagram. These diagrams were made for each of the 341 spans. The contour diagrams were prepared primarily for comparison with future resurveys. At this stage the major objective is to establish the general level of corrosion potential in a manner similar to the approach for visual observations and depths of cover.

The cumulative frequencies for all corrosion potential measurements, shown in Figure 15, indicate the proportion of spans which exhibited various levels of corrosion potential. Both the average corrosion potential and the maximum potential measurements recorded for the span are shown. As seen from Figure 15, approximately 95 percent of the spans had an average corrosion potential below 0.20 and 50 percent showed no individual value above 0.20. At the current level of development of the testing method, there is not general agreement as to the significance of specific levels of corrosion potential. It is generally agreed that values below 0.20 volt indicate no active corrosion while values above 0.40 volt indicate active corrosion. Because of uncertainties associated with the measuring techniques, as used in this study as

Bridge No. 64-002-102-46

Span No. 4

Length(s)

Grid Size 5' (1.5m)

Girder Type(s)

						←5'→							
						5'	.18	.12	.16	.14	.16	.12	.14
							.18	.18	.10	.10	.04	.10	.10
							.20	.20	.16	.10	.16	.10	.10
							.20	.20	.16	.18	.10	.04	.18
							.18	.18	.18	.06	.12	.08	.08
							.12	.18	.18	.16	.12	.12	.10
							.20	.12	.16	.16	.10	.10	.08
							.18	.22	.18	.18	.12	.08	.10
							.14	.12	.16	.16	.12	.06	.14
							.20	.20	.18	.20	.16	.04	.02
							.16	.18	.18	.16	.12	.16	.04
							.18	.18	.22	.18	.12	.14	.10
							.18	.16	.18	.10	.18	.18	.06
							.20	.20	.12	.20	.08	.12	.18

Figure 13. Corrosion values imposed on grid for the span.

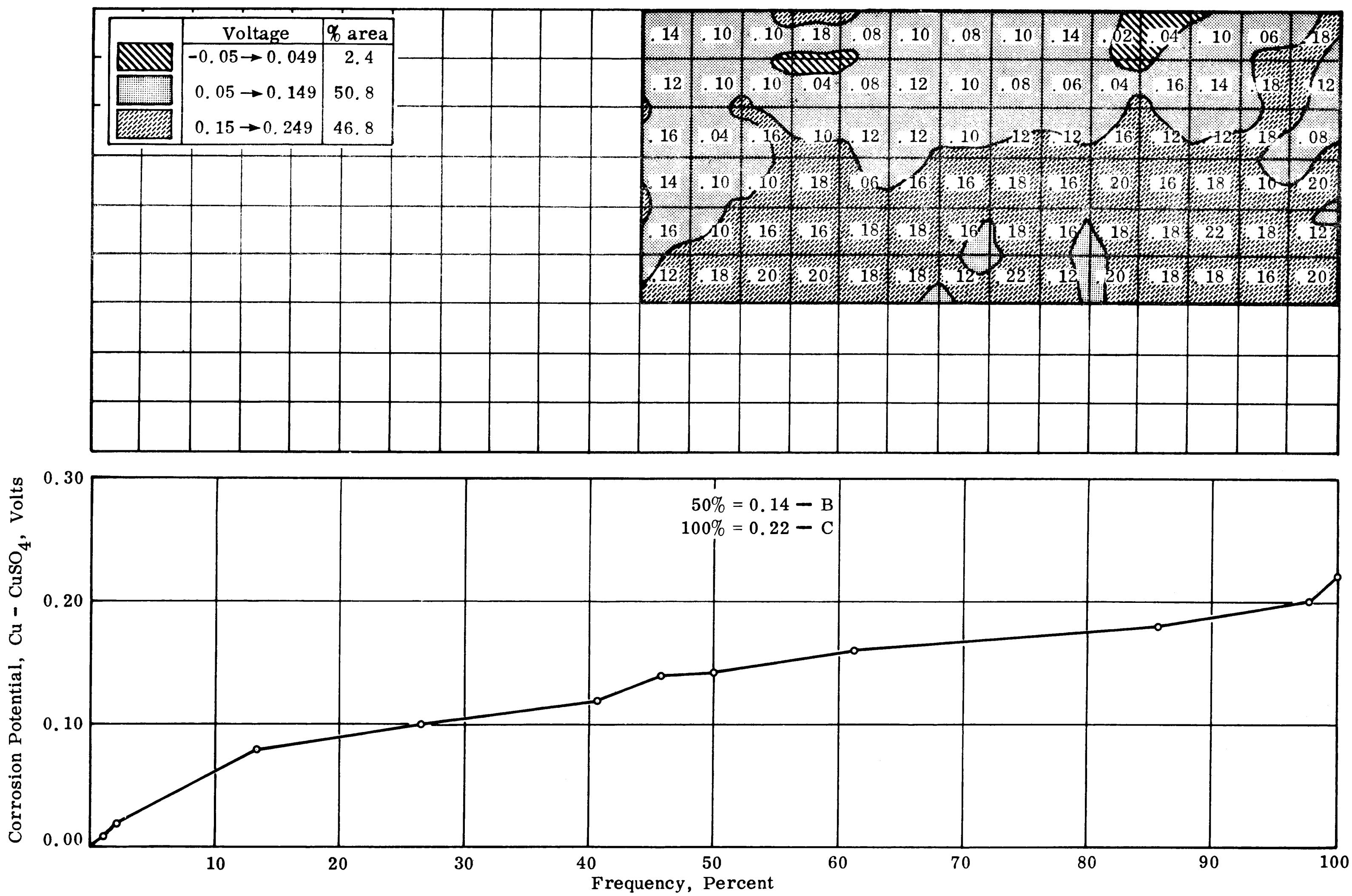

Figure 14. Equipotential and cumulative frequency diagrams for span in Figure 13.

2483

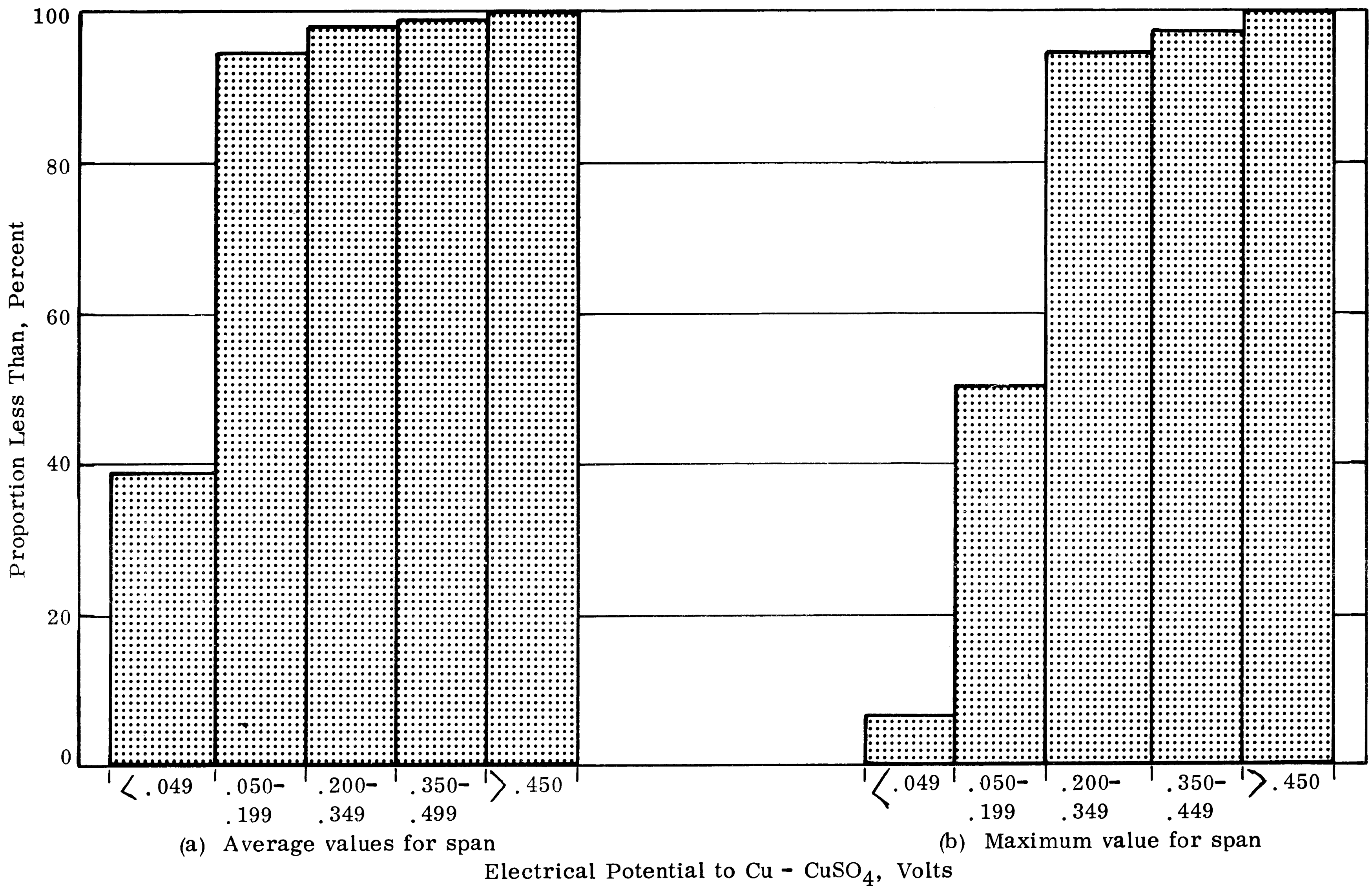

Figure 15. Cumulative frequency of measured corrosion potentials.

discussed later, it is perhaps prudent to take the values of 0.20 and 0.40 volt as the limiting values rather than 0.30 and 0.35 volt as suggested by Stratfull. The generally low levels of corrosion potential observed would be expected since the spans are comparatively new.

As noted earlier the measurements were made using the electrical ground given by the fortuitous connections between the reinforcement and exposed metal. The procedure obviously introduces some error, but the data probably reflect the proper relative orders of magnitude. It is hoped that future measurements made with the same ground connection will give a valid indication of the change in potential even though the exact magnitude of the indicated potential will be suspect.

A number of analyses were made to relate the corrosion potential to factors such as age, depth of cover, and traffic values, but no relationships were found.

During the course of the project measurements were made of corrosion potentials along with other methods for evaluating concrete in a limited number of decks undergoing repair. These decks were of course much older than those studied in the project. Detailed observations were made using three methods (electrical potential, chain drag, and hammer sounding). The results have been previously reported (Smith 1973). The principal findings and conclusions from that report are reproduced in Appendix D.

CONCLUSIONS

Based upon the results of this research, the following conclusions appear warranted.

(1) The frequency of early bridge deck scaling has been dramatically reduced by the upgrading of specification requirements and construction practices. Several specific changes such as increased air contents, use of linseed oil treatments as well as increased awareness of the problem all contribute to this improvement. Because concrete susceptible to scaling usually exhibits the defect at an early age this is an encouraging result. The elimination of premature scaling was a major target of the specification upgrading effort. The success of this effort is evident.

(2) Transverse and random cracking are indicated to be more frequent than before the upgrading. The reason for the increase in transverse cracking is not apparent and there is other evidence that the indicated increase in random cracking is related to closer observation and differences in classifications rather than to real causes. The severity of cracking does not seem to be serious enough to warrant attention. Real differences, if any, will become more apparent with time.

(3) The frequency of all other observed defects is very low. Based upon previous studies this will undoubtedly increase with age, traffic etc., but experience suggests that serious problems are indicated at comparatively early ages.

(4) The measured average cover over reinforcement is fortunately significantly greater than that specified. For the two levels of cover specified, 8 and 16 percent of the measurements are less than required. This is believed to reflect an acceptable level of control.

(5) Ninety-five percent of the spans have average corrosion potentials below 0.20 volt, which indicates no active corrosion. On one percent of the spans the average values are above 0.40 volt, which suggests the presence of active corrosion. The potential for corrosion will increase with age and exposure to deicing chemicals.

(6) The techniques developed for the BPR-PCA survey in 1961 and used in previous studies by the Research Council provide reproducible and useful evaluations of performance based upon visual observations. The procedures reflect general trends and levels as opposed to detailed causes and effects.

(7) When the bridges to be surveyed are similar in age and condition and when the sample is sufficiently large, observations on a single randomly selected span provide the same results as observations of all spans on the bridge. Stated in other terms, the observation of spans rather than bridges appears to be a valid approach.

RECOMMENDATIONS

(1) Because the level of the performance indicated has improved with respect to the deficiencies which were the objectives of the upgrading to current specifications and construction, and because the remaining defects continue to be infrequent in occurrence, the procedures for controls and acceptance of bridge deck concrete now in use should be continued.

(2) A resurvey of the bridges should be scheduled in 1977-78. The decks will then be five to ten years old.

ACKNOWLEDGMENTS

This project, which involved extensive measurements on bridges in the field under traffic, required the cooperation and attention of a variety of people from the Field Forces and the Research Council

The data concerning bridges from which the sample was drawn were provided by the Bridge Division, and for this special thanks are due T.J. Ogburn III. The scheduling of the inspections was arranged and coordinated by the eight district bridge engineers. Thanks go to them and to the variety of Residency personnel who actually provided the traffic control.

The visual observations were made by student assistants Michael North and Thomas Scott. Bobby Marshall and Lewis Woodson, materials technicians, were responsible for the corrosion potential and cover depth measurements. C.E. Smith directed the initial planning of the field operations.

The voluminous data were drawn by Frank Lee and Celik Ozyildirim, graduate assistants.

Appreciation is expressed to all of these and others who had lesser roles in the collection and analysis of the data from this project.

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APPENDIX A

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 A-1. One or more sheets were required for each bridge. Data describing each bridge included: state; county; highway number; survey bridge number; state bridge number; year built; type of deck covering, if any; type of deck repairs, if any; traffic volume (ADTC); use of air-entrained concrete; availability of construction records; span lengths; and span types.

A dual bridge was considered as two individual bridges. A widened bridge was either dropped from the survey or inspected only for information on the old portion.

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 A-1 70 percent of the area of span 4 had an average scaling condition classified as medium scale, and heavy scale was encountered in portions of the scaled areas.

The six classifications of cracking--box 1 for transverse; box 2 longitudinal; box 3, diagonal; box 4, pattern or map; box 5, "D"; and box 6, random--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 A-1, medium transverse cracking (box 1) was observed in span 1 of the bridge, heavy in span 3, and light in span 4. Random cracking (box 6) of the same severity was found on the same spans. There was no visible longitudinal (box 2), diagonal (box 3), pattern (box 4) or "D" (box 5) cracking in any spans.

The presence of any rust stains on the deck surface 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.

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 judgement of the inspector.

3491

R-300 (6/70; revised 6/72)

DATA SHEET FOR RANDOM BRIDGE SURVEY INSPECTION REPORT

State VIRGINIA County STAFFORD Route No. 3 Bridge No. 2003-089-102-02

Year Built 2-71 Location R.F.&P. RR.

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

ADTC
19160/2 = 9580

****

Classification of Deck Deterioration

Span Number		1 ✓	2 ✓	3 ✓	4	5	6
Length (feet) 140'		52'	73'	52			
Girder Type							
SCALING	(1) % Light						
	(2) % Medium						
	(3) % Heavy						
	(4) % Severe						
CRACKING	(1) Transverse	L	L	L			
	(2) Longitudinal						
	(3) Diagonal						
	(4) Pattern			L			
	(5) "D"						
	(6) Random	L		L			
RUSTING	(1)						
SURFACE SPALL (number)	(1) Small						
	(2) Large						
JOINT SPALL	(1) Expansion						
	(2) Contraction						
	(3) Construction						
POP-OUTS (number)	(1)						
PATCHED AREAS		1 joint spall patched					
GROUND AREAS							

Several long random + transverse cracks especially on span 3

Date of Inspection 6-23-72

* Rough finishing on deck reveals aggregate

Figure A-1. Typical survey form.

APPENDIX B

GUIDELINES DEVELOPED BY FHWA REGION 15 AND USED IN THIS PROJECT

LIST OF EQUIPMENT USED

The basic components of the steel corrosion detection device are commercially available and are listed as follows:

1. Two wire reels, containing 125 feet of No. 18 single wire and 300 feet of No. 18 single wire, respectively. These are available from the Agra Engineering Company, 551 South Quaker Street, Tulsa, Oklahoma 74120. Price $30 each.

2. Two 36-inch-long copper sulfate reference cells. These are available from the Harco Corporation, 4600 East 71st Street, Cleveland, Ohio 45216. Price $25 each.

3. Good quality D.C. Null voltmeter with accuracy 2 percent of fullscale capable of reading to ± 1 mv. The voltmeter we are using is a Hewlett-Packard model 419A, available from the Hewlett-Packard Company, 2 Choke Cherry Road, Rockville, Maryland 20840. Price $475.

4. Portable drill, 3/8-inch chuck capacity, self-energized, maximum no-load speed 900 rpm. The drill we are using includes a completely enclosed, removable 9-volt battery with capacity to drill at least 300 holes 1/2-inch diameter, in 2-inch thick dry fir. The drill is manufactured by the Black & Decker Manufacturing Company, 701 East Joppa Road, Towson, Maryland 21204. Price $129.

   It should be noted that the portable drill batteries are rechargeable and a battery charger should be purchased to complement the drill. The battery charger we are using costs $33 and is the Black & Decker Charger No. 949 available from the Black & Decker Manufacturing Company at the above-mentioned address.

5. Cover meter for determining the amount of concrete cover over reinforcing steel and the size of the reinforcing steel bars. We are utilizing a model C-4949 Pachometer, including probe and spacer, carrying case, instruction manual, and chest strap assembly. This was obtained from James Electronics Inc., Instruments Division, 4050 North Rockwell Street, Chicago, Illinois 60618. Price $575.

6. A good quality battery operated ohm-meter, capable of measuring from 0 to 200 meg-ohms with an accuracy of ±3%. The ohm-meter we are using is a Simpson Model 313 VOM, available from the Simpson Electric Company, Division of American Gage and Machine Company, 5200 West Kinzie Street, Chicago, Illinois 60644.

7. A 12-inch x 12-inch x 1/8-inch copper plate with a copper electrical connection and a non-metallic handle for convenience in moving the plate from point to point.

FIELD OPERATIONS - CORROSION DETECTION PHASE

1. DEFINITION OF TERMS

a. Corrosion - Oxidation of reinforcing steel.
b. Standard Half cell - A copper plate immersed in a saturated copper sulfate solution.
c. Potential - Level of the electrical charge.
d. Normal Potential - Any metal in water or water solution has a tendency to throw atoms into solutions as ions. There is an actual solution tension and level of electric charge will differ in degree with the position of the metal in the activity series.
e. Standard oxidation potentials for metals in a saturated solution of their own ions.

Metal	Reaction	E^o (Volts)
Zn	Zn^{+2} +2e-	+0.763
Fe	Fe^{+2} +2e-	+0.440
Cu	Cu^{+2} +2e-	-0.337

EXAMPLE: Fe - Cu = 0.440 - (-0.337) = 0.777 volts

f. Difference in potentials - The algebraic difference in potential of one metal from that of another.
g. Electromotive Series - Potentials of metals surrounded by a saturated solution of their own ions.

2. INTRODUCTION

In recent years premature concrete bridge deck deterioration has been reported with sufficient frequency to warrant modifications that will minimize such problems in the future.

Recent reports have identified concrete spalling as the most serious form of bridge deck deterioration, because of the severe effect it has on riding surfaces, the reduction in structural capacity and the difficulty in making a permanent repair. It has also become apparent over the past several years that the use of deicing chemicals has significantly accelerated the spalling process.

Research studies made by the State of California, Division of Highways, in cooperation with the Bureau of Public Roads, indicate most spalling of concrete bridge decks to be caused by corrosion of the reinforcing steel which can exert an internal pressure in excess of 4,000 pounds per square inch.

The purpose of steel corrosion detection tests is to:

a. Identify the cause of corrosion.
b. Provide a means for evaluation of repair methods.
c. Aid in evaluating preventive measures and design changes.

Proof of equipment performances will be shown by making electrical measurements on both old and new structures and by analyzing the concrete for salt content at various depths. In a few States the reinforcing steel will also be checked for visual evidence of corrosion.

3. TESTING INSTRUMENTS AND EQUIPMENT

The following equipment is being used by the Region 15 Corrosion Detection Team - other equivalent equipment would be adequate.

a. Hewlett-Packard D.C. null voltmeter.
b. Copper copper-sulfate half cell.
c. Two spools of No. 16 wire, one spool containing 100 feet of two-wire cable each connected to a jack on the spool for easy connection to the voltmeter and spring clips on the end for making connections to the reinforcing steel. The two wires are used to allow the changes in the field. (The two sizes of clips are necessary to allow easier connections to the reinforcing steel.) The other spool contains 300 feet of insulated No. 16 wire with a jack on the spool and a spring clip for attaching to the copper cooper-sulfate halt cell.
d. A hand drill, self-powered, with masonry bit.
e. Hand tools - hammer, chisel, files, etc.
f. An ohm-meter similar to Simpson No. 313 volt ohm-meter.
g. A 12-inch x 12-inch x 1/8-inch copper plate with clip for connecting the ohm-meter and means to connect a 36-inch handle - The entire bottom surface must be covered with sponges using wood dowel pins for connectors.

4. TESTING PROCEDURES

The following procedures should be followed when testing reinforced concrete bridge decks or continuously reinforced concrete pavements, for active corrosion of the reinforcing steel.

a. Measure and mark a five foot grid on the surface to be tested. (If conditions warrant, the grid may be increased or decreased.)

b. Locate a reinforcing bar or other connection to the reinforcing steel. A positive connection to the top of reinforcing steel is desired; however, if this is not feasible, the bridge railing expansion joints, light standards, drainage scuppers or other exposed steel may provide a positive connection to the reinforcing steel provided:

(1) The connection must not be galvanized.
(2) Checking the electrical level at various distances must show no constant decrease in electrical level.

c. Uncoil an ample length of wire to reach all areas to be tested, attach minus (-) jack of voltmeter to the reinforcing steel and plus (+) jack to the copper copper-sulfate half cell.
d. Check voltmeter battery for satisfactory charge.
e. Zero voltmeter on lowest scale.
f. Switch to WM-AM on the one (1) volt scale and make measurements of the electrical potential at each grid point. The half cell requires a wet sponge attached to the bottom contact to aid in making a good electrical contact with the concrete.

Potential readings from 0 to 0.30 volt are normal for sound concrete with no active corrosion in the reinforcing steel. When potential readings of 0.35 volt or more are encountered, the reinforcing steel is actively corroding.

g. Record the readings on graph paper and plot the lines of equipotential.

The following procedure should be followed when applying resistance tests on bridge decks with membrane water-proofing system.

a. Measure and mark a five-foot grid on the surface to be tested. (If conditions warrant, the grid may be increased or decreased.)

b. Wet the surface to be tested thoroughly and repeatedly allow the water time to permeate through the surface. The water should contain a wetting agent (95 ml of wetting agent to 5 gals water).
c. Locate a reinforcing bar or other connection to the top mat of the reinforcing steel. A positive connection to the top mat of the reinforcing steel is desired; however, if this is not feasible, the bridge railing, expansion joints, light standards, drainage scuppers or other exposed steel may provide a positive connection to the reinforcing steel provided:

Checking the resistance level at various distances along an exposed portion of the concrete must show a constant resistance level, thus indicating a positive connection to the reinforcing steel.

d. Uncoil an ample length of wire to reach all areas to be tested, attach the minus (-) jack of the ohm-meter to the reinforcing steel and the plus (+) jack to the 12-inch x 12-inch x 1/8-inch copper plate. Wet sponges.

e. Check ohm-meter battery for satisfactory charge.
f. Zero ohm-meter.
g. Switch to highest range of ohm-meter and record reading - if no reading is attained, switch to next lower range until a reading is attained. Reverse connections to meter and average the readings to reduce the error induced by galvanic coupling of the copper plate and the reinforced steel.

Resistance readings of bare concrete will vary from 1000 to 1300 ohms per sq. ft. Depending on the magnitude of the external galvanic voltages that exists, gross errors can occur in this low resistance range. For example, with the leads connected with one polarity the value can be in the order of 1000 ohms per sq. ft. By reversing the leads, the values can be in the order of 3000 or 4000 ohms per sq. ft.

It is speculated (according to California Study) that an excellent waterproof coating for bridges would always have an electrical resistance greater than 500,000 ohms per sq. ft., while a poor or perforated coating would never have a resistance greater than 100,000 ohms per sq. ft.

Note: For a more comprehensive study record readings of the corrosion detection device and the resistivity device.

h. Record the readings on graph paper and plot lines of equal resistance.

5. <u>PROCESSING AND REPORTING DATA</u>

Record the following data:

a. Location (route, nearest town and project number)
b. Type of construction
c. Year constructed
d. Number of spans
e. Major repairs
f. Span tested and date

Complete plotting equipotential or equiresistance lines, write a narrative including a statement on condition of the surface and your opinion as to whether active corrosion is present, or for resistance measurements a statement on apparent effectiveness of the membrane.

APPENDIX C--BRIDGES INCLUDED IN SURVEY

Random No.	Bridge No.	Route	County/City	Location	Description	Length
			(Bristol District)			
18	748	0460	148-101-01 Richlands	Clinch River	SS-IB-SC	210'
44	665	0602	098-130-10	Cripple Creek	SS-TA-SC	145'
55	663	0081	098-101-23	SBL Rts. 52 & 121 U	SS-IB-SC	284'
72	652	0613	095-148-16	Tumbling Creek	RC-SS-SC	64'
109	394	0621	052-124-07	Powell River	RC-SS-SC	181'
122	751	1319	164-131-08 Appalachia	Powell River	SS-TA-SC	118'
235	601	0058	084-701-00	Cove Creek	RC-SS-SC	23'
268	654	0710	095-129-12	South Fork of Holston River	SS-IB-SC	189'
404	653	0080	095-101-01	North Fork of Holston River	SS-IB-SC	312'
527	655	0803	095-140-15	Middle Fork of Holston River	SS-IB-SC	190'
554	612	0622	086-138-12	N & W Railway	SS-IB-SC	122'
654	136	0738	010-110-05	Walker Creek	RC-SS-SC	131'
746	613	0650	086-140-13	South Fork of Holston River	SS-TA-SC	130'
			(Salem District)			
76	444	0081	060-102-17	N & W Rwy. NBL	SS-IB-SC	242'
85	574	0690	080-141-24	Back Creek	SS-IB-SC	92'
93	448	0081	060-102-24	N & W NF Roanoke SBL	SS-DG-CC	323'
117	445	0081	060-102-18	N & W Railway SBL	SS-IB-SC	242'
138	562	0605	077-107-03	Little River	RC-IB-SC	197'
			(Lynchburg District)			
4	109	6029	005-106-11	739 over 29	SS-DG-SC	182'
5	111	0647	006-117-03	Falling River	RC-IB-SC	100'
28	344	0601	041-132-23	Hyco River	RC-SS-CC	157'
51	205	0607	019-123-15	Roanoke Creek	RC-IB-SC	157'
81	234	0696	024-110-04	Willis River	RC-SS-CC	91'
99	483	0639	062-128-18	Rockfish River	SS-IB-SC	235'
106	105	6029	005-106-02	Buffalo River SBL	SS-DG-SC	231'
112	553	0460	073-103-03	Sandy River	SS-IB-SC	113'

Appendix C continued

Random No.	Bridge No.	Route	County/City	Location	Description	Length
				(Lynchburg District cont.)		
113	546	0939	071-113-19	Sandy River	RC-IB-SC	173'
144	545	0903	071-169-21	Southern Rwy.	SS-IB-SC	157'
154	106	6029	005-106-06	Rt. 659 NBL	SS-IB-SC	145'
170	552	0460	073-103-02	Bush River	RC-IB-SC	188'
178	475	0056	062-108-06	Tye River	RC-IB-SC	144'
147	347	0740	041-135-27	N F & D Rwy.	SS-IB-SC	154'
236	484	0650	062-109-05	Rucker Run	RC-IB-SC	100'
241	555	0628	073-124-12	Briery Creek	SS-IB-SC	130'
258	209	0672	019-120-13	N & W Rwy.	RC-IB-SC	130'
355	346	0665	041-133-24	Sandy Creek	SS-IB-SC	130'
278	177	0020	014-101-01	James River & C & O Rwy.	SS-DG-SC	1007'
				(Richmond District)		
6	152	0085	012-101-10	Rt. 46 SBL	PS-IB-SC	170'
12	170	0085	012-101-30	Waqua Creek SBL	PS-IB-SC	228'
14	161	0085	012-101-19	19 N & W RR NBL	PS-IB-SC	159'
15	323	0064	037-101-04	Rt. 623 U	PS-IB-SC	308'
26	330	0064	037-102-12	Rt. 629 U	PS-DG-SC	297'
41	156	0085	012-101-14	Rt. 1 SBL	SS-IB-CC	333'
48	160	0085	012-101-18	Rt. 614 SBL	RC-IB-SC	120'
52	140	0046	012-091-600	Nottoway River	RC-IB-SC	434'
60	149	0085	012-101-07	Great Creek NBL	PS-IB-SC	148'
61	356	0738	042-160-12	North Anna River	SS-IB-SC	235'
64	253	0085	026-101-28	Hatcher Run SBL	PS-IB-SC	132'
83	428	0662	055-136-13	N & W Rwy.	SS-IB-SC	122'
87	218	0649	020-136-30	Falling Creek	RC-SS-CC	92'
89	169	0085	012-101-29	Waqua NBL	PS-IB-SC	213'
96	519	0460	067-101-02	Rt. 360 WBL		248'
98	247	0085	026-101-22	Gravelly Run Creek SBL	PS-IB-SC	141'
105	171	0085	012-101-31	Rt. 712 U	SS-IB-SC	499'
108	244	0085	026-101-19	Rt. 703 U	PS-IB-SC	341'
110	158	0085	012-101-16	Rt. 646 SBL U	PS-IB-SC	169'
121	163	0085	012-101-21	Rt. 642 NBL U	PS-IB-SC	157'

Random No.	Bridge No.	Route	County/City	Location	Description	Length
				(Richmond District continued)		
131	168	0085	012-101-26	Rt. 630 SBL U	SS-IB-SC	153'
134	362	0064	043-102-40	Rt. 271 U	SS-IB-SC	148'
136	239	0085	026-101-04	Rt. 650 SBL U	PS-IB-SC	
137	432	0001	058-000-02	Roanoke River	SS-TA-SC	
151	155	0085	012-101-13	Rt. 1 NBL	SS-IB-CC	350'
256	252	0085	026-101-27	Hatcher Run NBL	PS-IB-SC	132'
359	173	0085	012-101-34	Nottoway River SBL	PS-IB-SC	307'
				(Suffolk District)		
19	689	0301	109-102-01 Emporia	Meherrin River	SS-IB-SC	279'
21	706	0064	122-070-04 Norfolk	WBL over Little Creek Road	SS-IB-SC	428'
36	379	0625	046-135-05	Rattlesnake Swamp	RC-IB-SC	115'
68	628	0301	091-061-4	Nottoway River SBL	RC-IB-SC	
107	627	0301	091-061-2	Stony Creek SBL	RC-IB-SC	
128	721	0337	122-105-01 Norfolk	Lafayette River	SS-IB-SC	1755'
129	722	0337	122-105-02 Norfolk	Lafayette River	SS-IB-SC	1755'
146	723	0564	122-070-01 Norfolk	Granby Street EBL	SS-IB-SC	196'
181	704	0064	122-070-03 Norfolk	Little Creek Road EBL	SS-IB-SC	186'
285	702	0064	122-070-01 Norfolk	Granby Street EBL	SS-IB-SC	241'
611	385	0060	047-102-02	C & O Railroad WBL	SS-IB-SC	187'
				(Fredericksburg District)		
91	198	0654	016-126-11	Mattaponi River	SS-IB-SC	120'
347	620	0646	088-138-13	Matta River	RC-IB-SC	115'
494	622	6017	088-101-02	Over R F & P Rwy.	SS-IB-SC	156'
736	624	0003	089-102-02	R F & P Rwy.	SS-IB-SC	140'
				(Culpeper District)		
8	53	0064	002-102-28	Rt. 780 EBL	PS-IB-SC	148'
25	62	0064	002-102-37	Rt. 20 over Moores Ck. SBL	PS-IB-SC	141'
35	297	0660	032-103-02	Cuunningham Creek	RC-SS-CC	79'
40	292	0064	032-101-01	Rt. 799 EBL	PS-IB-SC	139'
43	68	0064	002-102-46	Rt. 22 & C & O Rwy. WBL	SS-IB-SC	364'
45	559	0675	076-153-14	Broad Run Creek	SS-IB-SC	160'
47	561	0776	076-152-13	Southern Rwy.	SS-IB-SC	160'
67	284	0644	029-168-21	Accotink Creek	SS-IB-CC	160'
69	671	0236	100-104-05 Alexandria	Ent. Cameron Station U	SS-IB-CC	210'

2500

Appendix C continued

Random No.	Bridge No.	Route	County/City	Location	Description	Length
				(Culpeper District continued)		
78	421	0064	054-101-11	Rt. 659 U	SS-DG-SC	297'
79	410	6007	053-111-16	Rt. 15 over Tuscr. Ck.	SS-IB-SC	48'
86	55	0064	002-102-30	Rt. 631 U	SS-DG-SC	334'
92	72	0064	002-102-51	Mechunk Creek EBL	SS-DG-SC	241'
102	674	7250	104-101-92	Rugby Avenue	SS-IB-SC	182'
			Charlottesville			
104	82	6029	002-106-08	Ex. Rt. 29 NBL	SS-IB-SC	178'
111	75	0064	002-102-63	Frontage Rd. EBL		113'
116	423	0064	054-101-13	Rt. 604	SS-IB-SC	379'
119	558	0616	076-134-07	Bull Run	RC-IB-SC	192'
124	231	6029	023-103-10	Southern Rwy. U.	SS-DG-SC	207'
127	224	0640	023-124-13	Thornton River	SS-IB-SC	190'
139	73	0064	002-102-52	Mechunk Ck. WBL	SS-DG-SC	241'
142	419	0064	054-101-09	Rt. 607 U	SS-IB-SC	275'
153	41	0064	002-102-13	Mechum River WBL	SS-DG-CC	254'
155	414	0064	054-101-04	Rt. 615 EBL	SS-IB-SC	149'
160	59	0064	002-102-34	Rt. 20 EBL	SS-DG-SC	344'
162	557	0253	076-101-01	Swk. U. Br. Rt. 95	RC-SS-SC	164'
267	400	0663	053-143-10	Catoctin Ck.	SS-IB-SC	220'
279	51	0064	002-102-26	Rt. 781 EBL	SS-IB-SC	150'
246	43	0064	002-102-15	Rt. 682 WBL	PS-IB-SC	122'
371	83	6029	002-106-09	Ext. Rt. 29 SBL	SS-IB-SC	194'
534	430	6029	056-107-01	Robinson R. SBL	SS-IB-CC	244'
632	524	0654	068-125-12	Pamunkey Ck.	RC-IB-SC	130'
638	429	0029	056-103-03	Crooked Run SBL	RC-IB-SC	128'
				(Staunton District)		
9	92	0311	003	Dunlap Creek	SS-TA-SS	109'
11	610	6211	085-102-01	Smith Creek	SS-IB-SC	180'
13	114	0612	007-154-23	South River	RC-SS-SC	131'

Appendix C continued

Random No.	Bridge No.	Route	County/City	Location	Description	Length
			(Staunton District continued)			
22	312	6037	034-101-06	Rt. 522 SBL	SS-IB-SC	283'
27	583	0609	081-124-05	Cedar Creek	RC-SS-SC	109'
31	311	6037	034-101-05	Rt. 522 NBL	SS-IB-SC	285'
56	93	0311	003-	Dunlap Creek	SS-TA-SC	125'
62	588	0033	082-101-08	Shenandoah River EBL	SS-DG-SC	427'
63	309	6037	034-101-01	Penn Rwy. NBL	SS-DG-SC	253'
65	527	0675	069-117-08	Shenandoah River	SS-IB-SC	510'
74	90	0064	003-104-44	Wilson Creek WBL	RC-IB-SC	129'
80	221	0007	021-102-03	Opequon Creek WBL	RC-IB-SC	240'
120	593	0617	082-126-15	North Fork of Shenandoah River	RC-SS-SC	217'
126	113	0275	007-101-01	Lewis Creek	RC-IB-SC	143'
130	694	U000	115-101-01	Cantrell Avenue C & W RR	SS-IB-CC	367'
			Harrisonburg			
132	680	0064	105-101-05	Jefferson Avenue U	SS-IB-SC	291'
			Clifton Forge			
133	529	7211	069-101-01	Hawks Bill Creek EB	RC-IB-SC	172'
149	313	6037	034-101-2C	SBL Rt. 11 NBL	SS-IB-SC	173'

APPENDIX D

PRINCIPAL RESULTS AND CONCLUSIONS FROM DETAILED STUDY OF ONE BRIDGE (SMITH 1973)

RESULTS

1 Percentage of deck area deterioration by Chain Drag Method:

Span 2	54%
Span 3	34%

2 Percentage of deck area deterioration by Hammer Method:

Span 2	74%
Span 3	48%

3 Percentage of deck area deterioration by Electrical Potential Method:

Span 2	65%
Span 3	74%

4 Percentage of area of corrosion to deck replaced as defined by the Hammer Method:

Span 2	33%
Span 3	23%

5 Percentage of total area of corrosion in area defined by the Chain Drag Method:

Span 2	62%
Span 3	73%

6 Percentage of total area of corrosion in area defined by the Electrical Potential Method:

Span 2	71%
Span 3	80%

7 Percentage of area of corrosion to area of agreement by all methods:-

Span 2	53%
Span 3	45%

CONCLUSIONS

An attempt has been made to show the degree of agreement between the indications from the three most widely used methods for detecting bridge deck deterioration associated with spalling. This was done by using the three methods to survey two spans of a bridge that was scheduled for deck replacement, drawing scale layouts of deteriorated areas as indicated by the methods, and superimposing these layouts so that areas of agreement could be found. A visual survey was conducted on the reinforcing steel exposed during the replacement operation as a basis for evaluating the effectiveness of the detection methods. The following general conclusions can be drawn from this project:

1. The three techniques were found to be practical and effective.

2. It was concluded from comparing the potential survey results with the results of the visual survey that high potential readings in a large percentage of the areas relate to corrosion of the reinforcing steel. However, it was noted in some instances that this was not true.

3. To ensure that a high percentage of the deteriorated areas of a deck are located, two of the detection methods should be used and the areas indicated by both methods should be removed.