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LONG-TERM PERFORMANCE OF CONCRETE CONTAINING HYDRATED HYDRAULIC LIME AS AN ADMIXTURE

by

Howard H. Newlon, Jr.
Associate Head, Virginia Highway & Transportation Research Council

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

Virginia Highway & Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia Department of Highways & Transportation and the University of Virginia)

In Cooperation with the U. S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

December 1976
VHTRC 77-R32

2702

SUMMARY

In 1941 two bridges were constructed with decks using concrete incorporating as an experimental feature the use of hydrated lime as an admixture in amounts varying from zero to 12%. After 35 years of service the decks were scheduled for repairs to correct surface spalling caused by corrosion of the reinforcing steel. Performance surveys were made including the measurement of electrical corrosion potential. Cores were removed for petrographic examination, chloride content and analysis by X-ray diffraction.

Based upon these observations it was concluded that:

1. The performance of the concrete in the decks has been excellent considering the age of the structure and the lack of air entrainment. There is no significant scaling or other general deterioration.

2. The major defect is surface spalling from corrosion of the reinforcement. This is not surprising in view of the high chloride levels and comparatively shallow concrete cover above the top reinforcement

3. Based upon electrical potential measurements, the corrosion potential decreases with increasing additions of hydraulic lime, for additions of 4% and 8%, and increases slightly between 8% and 12%. If this reduction is real, it must be related to some initial passivation of the reinforcement from either the lime itself or included alkalies that over the years was overcome but which still is reflected in the extent of corrosion.

4. The overall excellent durability of the concrete in these decks as compared with the concrete in contemporary decks undoubtedly reflects the greater than average testing and inspection exercised in connection with the experimental features of the project.

5. Within the limits of the chemical and petrographic analyses, no unusual reaction products or characteristics were observed among the several types of concrete used.

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INTRODUCTION AND BACKGROUND

The use of admixtures to modify the properties of freshly mixed and hardened concrete is almost as old as the use of concrete itself. Marcus Vitruvus Pollio, in the first century A.D., recorded that an essential ingredient in his recipe for hydraulic cement mortar was either hog's lard, curdled milk, or blood (Mather 1967). In the light of current understandings of concrete technology, it can be said that these materials undoubtedly acted to entrain air in the mixture with concomitant improvements to workability and placeability. Among the numerous advances in concrete technology that have been made during the twentieth century, none is more important to the production of durable pavement surfaces than the discovery that purposefully entrained air improves workability, reduces segregation and provides resistance to freezing and thawing, particularly in the presence of deicing chemicals. As a result of this discovery, or to some extent, the rediscovery of air entrainment during the early 1940's, concrete placed in modern pavements and bridge decks universally contains air entraining admixtures. Subsequent to the acceptance and use of air entraining admixtures, rapid progress was made in developing other admixtures for reducing the water requirement of concrete or controlling its setting time. ASTM Designation C494 "Standard Specification for Chemical Admixtures for Concrete" covers five types of such admixtures intended to provide various combinations of water reduction and set modification. The use of these and other mineral admixtures is widely practiced (ACI Committee 212 1971). Current research and development is being directed toward a class of admixtures described as "superplasticizers," which greatly reduce the water necessary to produce a workable concrete mixture.

The first use of air entrainment for pavements in Virginia was in the construction of the Shirley Highway in Northern Virginia in 1950 (Furgiuele and Melville 1952). The initial experimental use of air entrainment in a bridge was in the Westham Bridge in Richmond in 1949 (Melville 1949). Other states, particularly those in the northern areas, adopted air entrainment earlier than Virginia, a fact that later contributed to a comparatively high incidence of deicer scaling in the state.

Admixtures for reducing the water requirement or modifying the setting characteristics of concrete have also been studied by the Research Council (Newlon and Morgan 1959; Newlon 1961).

The general acceptance of admixtures is reflected in the fact that current specifications of the Virginia Department of Highways and Transportation require that all concrete be air entrained and that concrete used in bridge decks contain a water reducing or water reducing-retarding admixture unless its deletion is justified by job or weather conditions.

The important influence of the water cement ratio on the properties of concrete was reported in the classic work of Abrams (Abrams 1918). During the period between Abrams' work and the commercial development of air entraining admixtures, the need for placement of low slump mixtures was emphasized and the need for improvements in the workability of such mixtures increased.

Among the admixtures that received major attention during this period was hydrated, or to a lesser extent, hydraulic, lime.

Quicklime (CaO) is the product of calcination of a source of calcium carbonate, usually limestone or shells. For ease of handling and safety quicklime is usually "slaked" by the controlled exposure to water. The result is "slaked" or "hydrated" lime $Ca(OH)_2$. Hydrated lime is the active ingredient in most lime mortars. These mortars hardened by virtue of combination of the CaO with atmospheric CO_2 to form $CaCO_3$. When the source of lime contains a significant amount of clay minerals, by virtue of the presence of alumina and silica, the resulting lime usually exhibits the characteristics of a hydraulic cement, namely the ability to harden under water. Thus all commercially used lime is hydrated and some additionally would be characterized as hydraulic. A natural cement may be thought of as a hydraulic lime with a high cementing action.

When used as an admixture for concrete, lime was added with the cement or dissolved in the mixing water. Like many of the developments in concrete technology the benefits of using lime as an admixture were not universally accepted. An early trade publication of the National Lime Association (1926) stated that "The best way to make concrete water tight is to add hydrated lime to the mix. It is also the cheapest method..... No other material combines the qualities of permanence and void filling, with high workability and uniformity, freedom from segregation and cracking, high strength and pleasing color." Earlier, in 1912, Ernest L. Ransome, a pioneer in reinforced concrete construction, wrote: "Of other additions to Portland Cement with which I have experimented I must mention lime and clay. The former addition is so liable to abuse that I have largely abandoned it, except for the construction of waterproof tanks, and even then it is not indispensable. The fact seems to be that an addition of from three to five percent of slaked lime is beneficial when added as 'milk of lime,' using the limey water from mixing instead of plain water; the trouble arises as soon as lumps of lime putty, however small, find their way into the concrete, or when the amounts exceeds five percent" (Ransome 1912).

The 1926 National Lime Association publication provides numerous photographs and testimonials to the benefits of hydrated lime with major emphasis upon the improved watertightness as compared with concrete not containing lime as an admixture. It is interesting to note that the characteristics attributed to the lime admixture are almost exactly the same as those provided today by air entraining admixtures as evidenced in a 1928 report by Butcher, which stated:

> Concrete to which hydrated lime has been added is sticky enough to stand up without slumping, and at the same time sufficiently slippery to facilitate the working of the mass so that it will flow into the moulds quite easily, or into the forms where concrete is cast in situ. This increased plasticity also enables the mixed concrete to be conveyed through chutes without clogging, a good 'flow' being obtained without the use of too much water, which would be detrimental to the ultimate strength of concrete. Harsh dry mixtures are always difficult to place economically and are often dosed with water and almost washed into place. (Butcher 1928)

Prior to World War II, a number of reports were published, most of which dealt with hydrated lime, that was commonly available and widely used in the building industry (Hutchinson 1931; Antill 1935; Fisher 1941). There are fewer published reports on studies of hydraulic lime used as an admixture. One of the more extensive studies was reported from tests made by a large Virginia producer of hydraulic lime by Hutchinson (1931). These studies utilized a commercially available hydraulic lime that was marketed throughout the state. Although the material was widely used in the building trades, its use by the Highway Department was limited. There is no mention of its use in specifications of the Virginia Department of Highways.

Because of its cementing properties, hydraulic lime was believed to offer the benefits of hydrated lime plus additional cementing action from its own hydraulic activity. Hutchinson showed that for an increase in the water/cement ratio of 0.20 by vol. (.06 by wt.), strength was reduced by 900 psi (6.2 MPa), from 3,200 to 2,300 psi (22.1 to 15.9 MPa), for concrete made with portland cement while for equivalent mixtures containing an admixture of hydraulic lime, the corresponding strength reduction was only 500 psi (3.5 MPa). During the 1930's several large bridges were built under city jurisdictions that required the addition of masonry cement (hydraulic lime) to the mixture.

In 1940-41 a major crossing of the Shenandoah River, consisting of two bridges, was constructed at Front Royal. One structure, 1,090 ft. long (332 m), spanned the North Fork of the Shenandoah; and the other, 1,924 ft. long (586 m), spanned the South Fork. A major feature of this project was the inclusion of test sections incorporating various percentages of hydraulic lime. The research was cooperative with the Public Roads Administration (predecessor of the FHWA). The details of the project are given later in this report. There is no indication that hydrated/hydraulic lime was subsequently specified for use by the Virginia Department of Highways. No published reports of the project have been found. It is probable that the interruption in Departmental operations occasioned by World War II and the subsequent general acceptance of air entrainment, which provided many of the same potential benefits, account for the lack of continuing interest in the project. For the same reasons, it appears that following World War II the use of hydrated lime as an admixture declined nationwide. It is interesting to note that throughout this period some agencies required that a portion of the cement used in concrete be natural rather than Portland Cement. Notably, the state of New York continued this practice into the 1960's.

CHARACTERISTICS OF ORIGINAL CONSTRUCTION AND RESEARCH

As noted earlier, two structures were included in the Shenandoah River project. A plan view of the bridges is shown in Figure 1. The slabs are carried by deck trusses on concrete piers mostly spaced 120 ft.-8 in. (36.8 m) apart. The combined length of the bridges which were joined by approximately one-half mile (805 m) of pavement, was approximately 3,000 ft. (914 m). The decks contained 36 placement units, of which 24 were designated as experimental and the remaining 12 as control or standard placements; the experimental feature being the amount of hydraulic lime used as an admixture. The 36 sections are indicated in Figure 1, the mixtures, summarized in Table 1, were described as follows:

Class A. Standard Class A concrete as specified in Section 336.05.

Class A-4. Class A concrete in which 4 percent of the cement by weight will be replaced by an equal absolute volume of hydraulic lime.

Class A-8. Class A concrete in which 8 percent of the cement by weight will be replaced by an equal absolute volume of hydraulic lime.

Class A-12. Class A concrete in which 12 percent of the cement by weight will be replaced by an equal absolute volume of hydraulic lime.

Class A-4A. Class A concrete to which hydraulic lime will be added in an amount equal to 4 percent of the absolute volume of the cement.

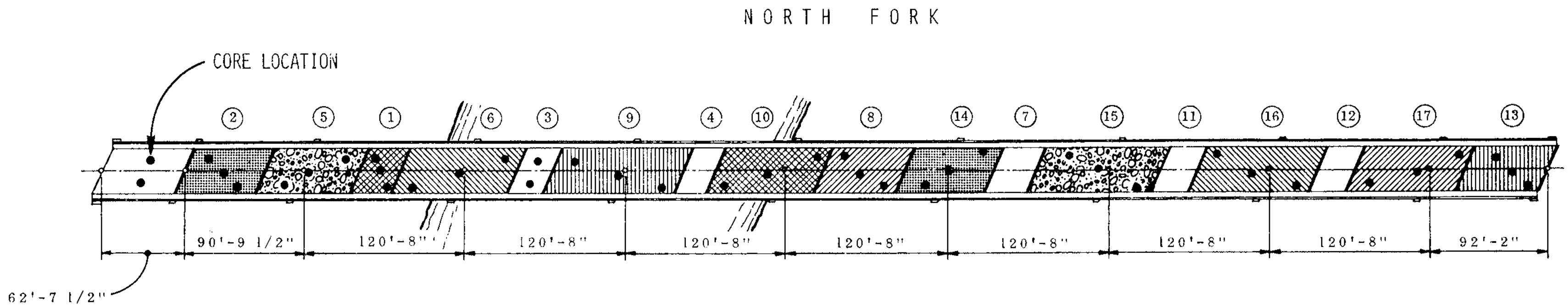
Class A-8A. Class A concrete to which hydraulic lime will be added in an amount equal to 8 percent of the absolute volume of the cement.

The cement content of the Class A concrete was calculated to be 6.15 bags or 565.4 lb. per cubic yard (335.5 kg/m^3) and the maximum water/cement ratio was 6.00/gal./bag or 0.53 by wt. All concrete was non-air entrained. The characteristics of the test sections are summarized in Table 1. The locations of the test sections are indicated in Figure 1.

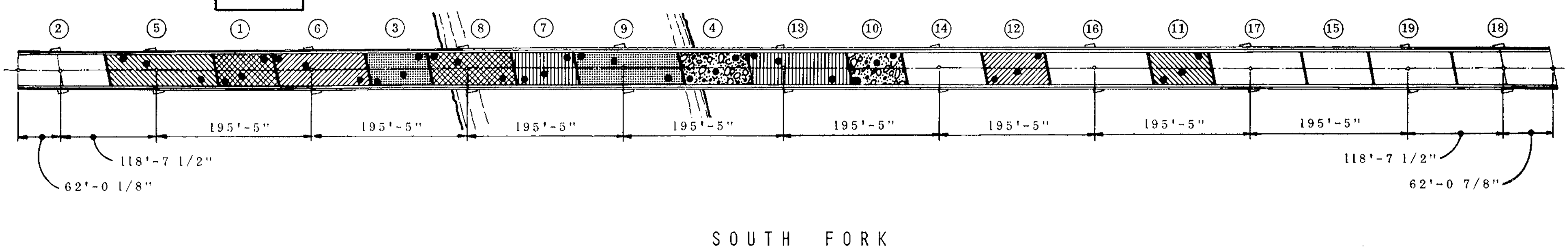
Only a few original records relating to the original project were found. Three items are reproduced as Appendices A, B, and C. The special provisions for the project are included in Appendix A and pertinent parts are summarized below. Descriptions of records, organization, and concrete control are given in Appendix B. None of the records described were found. In Appendix C results from strength tests representing 23 days of construction are given. Results for seven days (July 11, 12, 14, 15, 16, 17, and 18, 1941) relate to the deck placements, which are the major focus of this report. The pertinent strength data are discussed later.

The intended mixture proportions for the several classes of concrete are summarized in Table 2, based upon the data given in Appendix A. Calculated values of total cementing material are also shown. The actual mixture proportions are not available and it is probable that the water content was less in some cases than the maximum allowable value shown.

The comparative cementing capabilities of the portland cement and the hydraulic lime used are unknown, but the total range of cement plus lime from lowest to highest is approximately 5%, a relatively insignificant difference. It is likely that any improvement of properties or subsequent performance was probably related to the reduction in actual water requirement or improved compactibility rather than to the lime content per se.



DESIGNATION	POUNDS OF LIME PER 94 LB. PORTLAND CEMENT	SECTIONS NORTH FORK	SECTIONS SOUTH FORK
A	0	1,10	1,8
A4	3.3	2,14	3,9
A8	6.8	5,15	4,10
A12	10.6	6,16	5,11
A4A	3.1	8,17	6,12
A8A	6.2	9,13	7,13
NONEXPERIMENTAL	0	3,4,7,11,12	2,14,15,16,17,18,19



1 FOOT = 0.3048 METRES
1 POUND-MASS = 0.4536 KILOGRAMS

Figure 1. Location of test sections and cores.

Table 1

Description of Experimental Sections

Section Number	Floor Slab Class of Concrete	Character of Construction
	South Fork — Project 1015-B1	
1 and 8	A	Experimental
3 and 9	A-4	"
4 and 10	A-8	"
5 and 11	A-12	"
6 and 12	A-4A	"
7 and 13	A-8A	"
2-14-15-16-17-18-19	A	Nonexperimental
	North Fork — Project 1015-B2	
1 and 10	A	Experimental
2 and 14	A-4	"
5 and 15	A-8	"
6 and 16	A-12	"
8 and 17	A-4A	"
9 and 13	A-8A	"
3-4-7-11-12	A	Nonexperimental

Table 2

Intended Mixture Proportions

Class	Quantities of Materials per sack (94 pounds of Portland Cement)*			Calculated Values		Approximate yield* ft.^{3}/sack*
	Cement, lb.	Lime, lb.	Water, gal.	Cementitious Material (Portland Cement plus limes) lb/yd^3*	Ratio of water to cementitious material (by wt.)	
A	94	0	6.00	589	0.53	4.31
A-4-A	94	3.1	6.20	601	0.53	4.36
A-4	94	3.3	6.21	586	0.53	4.48
A-8A	94	6.2	6.40	615	0.53	4.40
A-8	94	6.8	6.43	583	0.53	4.67
A-12	94	10.6	6.68	579	0.53	4.88

*Taken from extant project records.

pound-mass = 0.4536 kilogram

gallon (U.S. liquid) = 0.003785 liter

pound-mass/cubic yard = 0.5933 kilogram/cubic meter

cubic foot = 0.02832 cubic meter

Based upon the strength data given in Appendix C, it is apparent that the concrete control on the project was excellent, even by modern standards. The seven days of deck placement are represented by 12 sets of 3 cylinders and 1 set of 2 cylinders. Each of these sets apparently was taken from a single batch. There is, unfortunately, no way to relate the sets to a specific variable. The strength results are summarized in Table 3.

Table 3

Compressive Strength Data at Time of Construction

Set	Average 28-day Compressive Strength, psi*	Standard Deviation, psi
1	5,287	186
2	5,110	170
3	5,010	17
4	5,030	165
5	4,457	289
6	5,480	419
7	5,343	289
8	5,420	299
9	5,745	21
10	5,290	101
11	5,257	49
12	5,210	201
13	4,577	430
All cylinders	5,161	389
	Coefficient of Variation	7.5%

*pound-force/square inch (psi) = 0.00689476 MPa.

According to standards established by the American Concrete Institute, coefficients of variation below 10% indicate excellent control (ACI Committee 214). The results undoubtedly reflect the extensive control effort described in Appendix A.

PERFORMANCE OF DECKS

The bridges were opened to traffic in 1941. The traffic characteristics at 10-year intervals throughout their service lives are indicated in Table 4.

The bridge over the North Fork had a 34-ft. (10.4 m) clear roadway, while the roadway of the South Fork structure was 40 ft. (12.2 m). Thus, the traffic was more highly channelized on the narrower bridge.

Table 4

Traffic Data
(Average Number of Vehicles per 24 Hours)

Year	Passenger Cars	Trucks and Buses	TOTAL
1944	1,337	434	1,771
1954	6,201	1,386	7,587
1964	7,750	1,235	8,985
1974	10,300	3,415	13,715

Based upon previous studies by the Research Council (Davis, North and Newlon 1971), the geographic area in which the structures were located would be expected to have approximately 50 cycles of freezing and thawing annually. The decks were subject to periodic chemical deicing. The frequency and amount of deicing have increased significantly throughout the life of the decks. It is important to note also that the decks probably had opportunity to mature before being exposed to deicing chemicals as opposed to current conditions.

In general, the performance of the decks was above average for structures of the same age, traffic characteristics and environment. A report by the district bridge engineer preparatory to advertising the contract for repairs indicated that the relative soundness of the concrete in these bridges was superior with respect to durability to that of many other bridges built in the early 1940's. He noted that there was some light scaling, particularly in the gutter areas, and a considerable amount of deck spalling. The latter defect he attributed to insufficient cover over the top reinforcing steel. This impression of better than average performance was shared by many people throughout the Department. With the view that the improved performance might be related to the experimental features of the original construction, a general consensus developed that the concrete should be studied to determine any evident cause-effect relationships.

Prior to the major rehabilitation efforts initiated in 1974, the decks had undergone one major repair effort. Prior to 1970, minor patching was accomplished by state forces. In 1970-71, extensive patching of spalled areas of the type later described was accomplished, again by state forces. In the 1950's the decks were overlaid with a light sand asphalt mixture as part of the statewide deslicking effort. The major repairs in 1970-71 were to correct surface spalling associated with corrosion of the upper reinforcing steel. In recent years such spalling has received major attention nationwide, although studies by the Research Council (Davis, North, and Newlon, 1971; Newlon 1974) have indicated that the defect is not widespread in Virginia; however, where it does occur, it poses a serious and costly repair problem.

RESEARCH STUDIES — 1974

The observations and sampling were conducted during a major repair contract that was awarded to correct defects associated with surface spalling that continued to develop subsequent to the 1970-71 patching. The areas to be repaired were identified by tapping with a hammer. These were then marked, removed, and patched with air entrained portland cement concrete. Generally, repairs proceeded sequentially along a single lane of both bridges and two-way traffic was maintained on the remaining lanes. The testing and sampling were conducted simultaneously with the marking, removal and repair procedures under way within the areas where traffic control was already established. This situation and the necessity for minimal interference with the contractor's operations in a few cases compromised the research studies, but, in retrospect, probably did not significantly affect the results obtained. A general view of the operation on the South Fork structure is shown in Figure 2 and a close-up of a typical area needing extensive repairs is shown in Figure 3. The areas identified as needing repair varied considerably from span to span; for example, on the South Fork bridge, of the 98 sections (original placements), 24, or about 25% were judged to need no repairs. As is evident in Figures 2 and 3, the repaired areas were predominantly in the wheel paths. This condition is discussed later in the report.

As outlined in the work plan for the project (Newlon and Walker 1974), the purpose of the research was to evaluate the concrete with regard to a possible relationship between its performance and the amount of hydrated lime present.

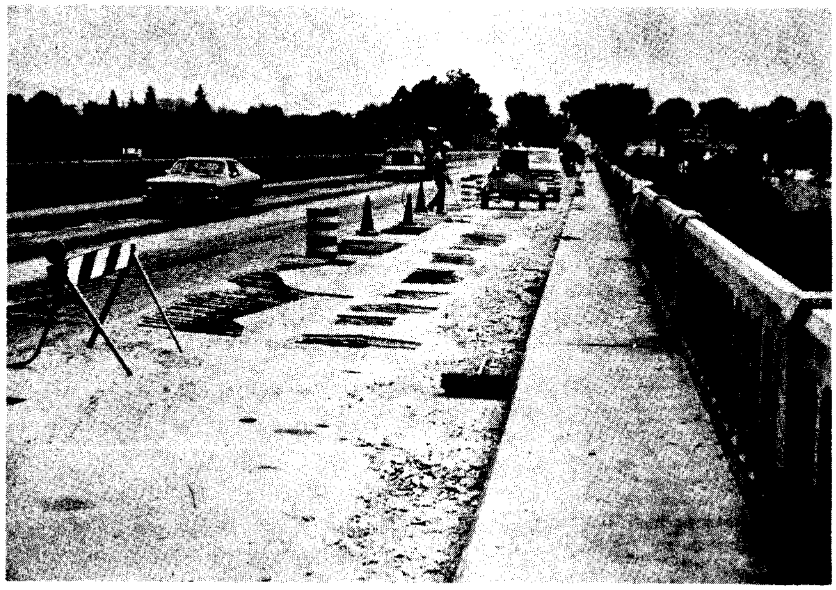


Figure 2. General view of South Fork structure.

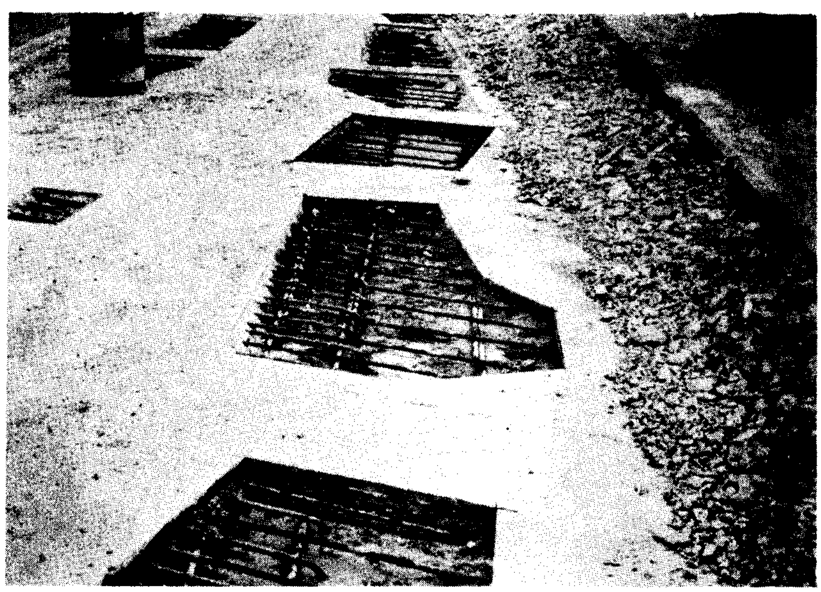


Figure 3. Close-up of area needing extensive repairs.

The tests and observations included:

1. A visual survey to determine the extent of various defects,

2. determination of depth of cover of the upper reinforcement,

3. petrographic examination of cores from each of the experimental and four of the nonexperimental sections,

4. chemical analyses for chloride content, and

5. X-ray diffraction studies to determine differences in the amounts of lime or the presence of any unusual reaction products.

For the purpose of identifying sections and samples, the South Fork structure was designated "S" and the North Fork structure "N". The lanes were identified as "A, B, C, or D" as indicated in Figure 1. The North Fork structure carried 3 lanes so that "A" indicated the northbound lane (NBL); "B" the center or passing lane (PL); and "C" the southbound lane (SBL). The South Fork structure carried 4 lanes, so that "A" designated the northbound traffic lane (NBTL), "B" the northbound passing lane (NBPL), "C" the SBPL, and "D" the SBTL. The section number corresponded to the sections shown in Figure 1. Thus, sample S-11-A designated a core from the NBL of the South Fork structure in section 11. Section 11 contained 12% added lime.

Visual Condition Survey

The surface condition of each section was observed and all defects recorded. While the frequency of defects varied from section to section, the only defect of significance was surface spalling. An area typical of moderate to severe spalling is illustrated in Figure 4. The area shown had been marked for removal and repair. In recent years, this type of spalling has received major attention in the technical literature. Opinions vary as to the mechanism involved in the development of such spalling, but it is generally agreed that the spalls develop from corrosion of the reinforcement that results from penetration of chloride ions through essentially sound concrete. The cracking associated with spalling results from corrosion, rather than forming initially and providing access of the chloride ions to the reinforcement which then corrodes. In most cases a separation develops at the level of the upper steel. This separation, called a fracture plane, radiates outward and upward. When it reaches the

Figure 4. Area of moderate to severe spalling.

surface an incipient spall exists, and the action of traffic dislodges the material (NCHRP 1970). The condition is clearly indicated in Figures 5 and 6, which show a location where two cores were removed from the SBL of the North Fork bridge. The horizontal crack is evident in both core holes as seen in Figure 5, and the nature of the separation is clearly shown in Figure 6. Above and below the separation, the concrete was sound, tough, and of excellent appearance. The entire surface of both structures was remarkably free of surface mortar deterioration (salt scaling), and had experienced only minor abrasion from traffic. These findings are particularly unusual in view of the use of non-air entrained concrete. However, from numerous studies nationwide, it has generally been observed that surface spalling and salt scaling rarely occur on the same deck (NCHRP 1970). Corrosion of reinforcement was always associated with the occurrence of spalling.

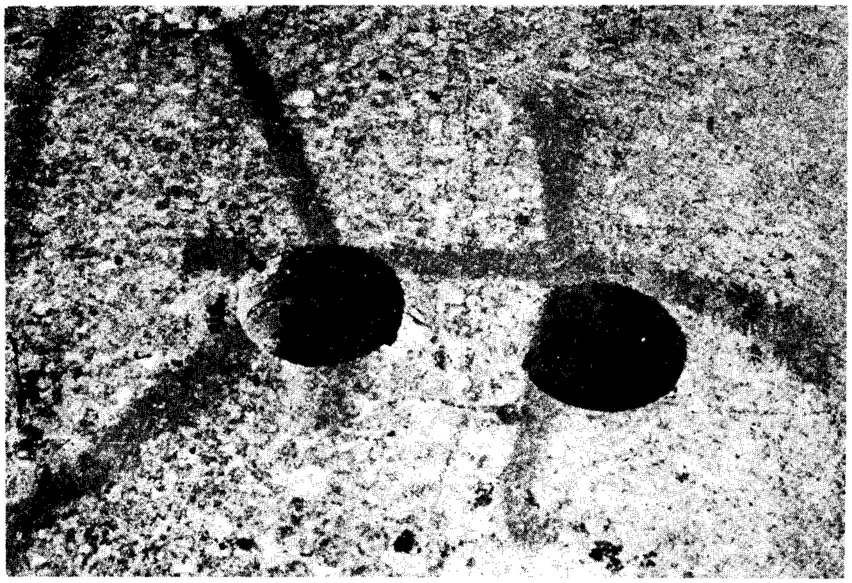


Figure 5. Core holes showing incipient spall.

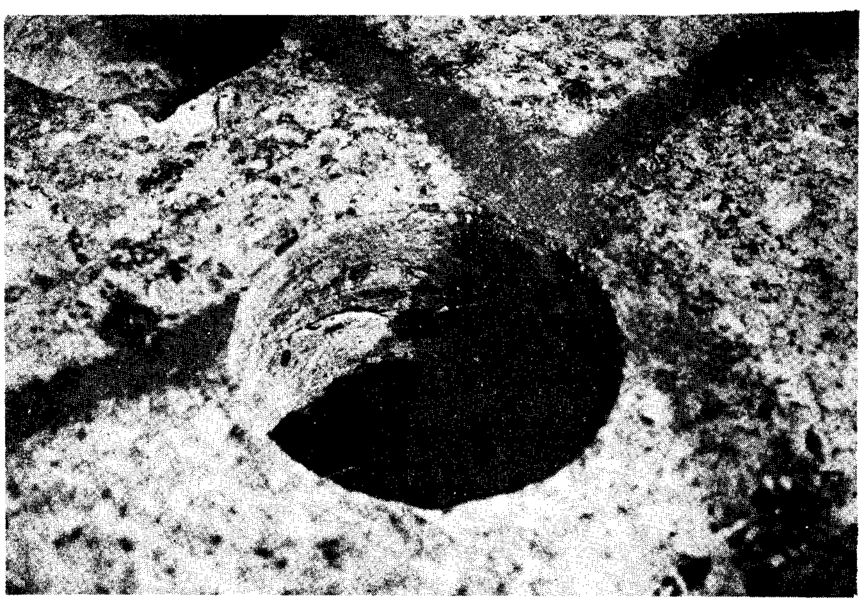


Figure 6. Close-up of incipient spall in Figure 5.

A view of one of the large repair areas is shown in Figure 7. A close-up of the reinforcement is shown in Figure 8. It can be seen that on a given bar, the corrosion is not general but rather occurs in isolated spots immediately adjacent to unaffected steel. This behavior has also been generally observed nationwide, and is attributed to the fact that the corrosion results from the creation of galvanic cells in local areas due to differential concentrations of chloride ions at various locations along the reinforcing bar.

The areas indicated as needing repair were almost completely confined to the wheel paths. The reason for this is not clear. There has been speculation, but no proof, that the pressure under tires accelerate the penetration of chloride-bearing water in these areas. A more plausible explanation is that the action of tires in these areas accelerates the propogation of fracture planes by the mechanism previously described.

As noted earlier, spalling of the type illustrated in the figures occurred on about 75% of the decks. No other defect of significance was observed. Extensive patching had been done in 1970-71. The quality of these patches and the workmanship was unusually good, to the point that the patched areas were often difficult to locate. In 1974, the areas designated as needing repair often went just to the formerly repaired areas as shown in Figure 9.

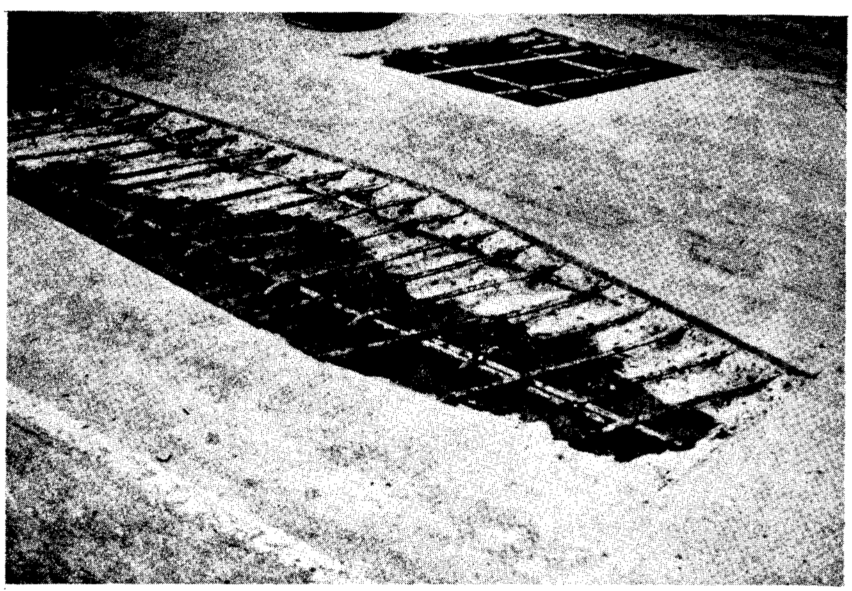


Figure 7. Large repair area.

Figure 8. Close-up of reinforcement showing variable corrosion.

Figure 9. Area designated for repair adjacent to area repaired in 1970.

Depth of Cover

On the South Fork decks, the depth of cover was determined with a pachometer at approximately 24 locations on each experimental section, which resulted in 840 measurements. On the North Fork decks, cover measurements were made at approximately 10-ft. (3.05 m) intervals along a line parallel to the centerline of the bridge. These measurements provided 208 values. Since no systematic relationship was expected or found between the study variables and the depth of cover, these data are meaningful only in terms of providing an overall indication of the depth of cover associated with the construction process. The results of these measurements are presented in Table 5.

Table 5

Depths of Concrete Cover Above Top Reinforcement

Bridge	Number of Measurements	Average Clear Cover, $\bar{X}$, in.	Standard Deviation, S, in.
South Fork	840	1.79	0.41
North Fork	208	1.87	0.24

Note: 1 in. = 25.4 mm.

The distributions of measurements for the two decks are presented in Figures 10 and 11. For these decks, the specified distance between the deck surface and the center of the uppermost steel was 1-3/4 in. (44.5 mm). This would be approximately 1-7/16 in. or 1.31 in. (33.3 mm) of clear cover. From the distributions in Figures 10 and 11 it can be seen that in the case of the South Fork decks, 18% of the measurements were less than the specified value, while in the North Fork structure, 7 were less than the specified. By modern standards, a minimum value of 1-7/16 in.(33.3 mm) is not considered adequate with respect to protection from spalling, but is close to the "breaking point" of the data developed in Kansas (Crumpton et al. 1969). Today, it is generally agreed that a clear cover of at least 2 in. (50.8 mm) is required to provide protection against corrosion. In a recent statewide study (Newlon 1974), the average clear cover determined from 10,170 measurements on 339 decks was 2.40 in. (61 mm). Interestingly enough, the standard deviation from that study was 0.49 in. (12 mm), approximately the same as that found on the South Fork structure in these decks constructed in 1941. It is clear that the comparatively shallow cover strongly influenced the occurrence of corrosion.

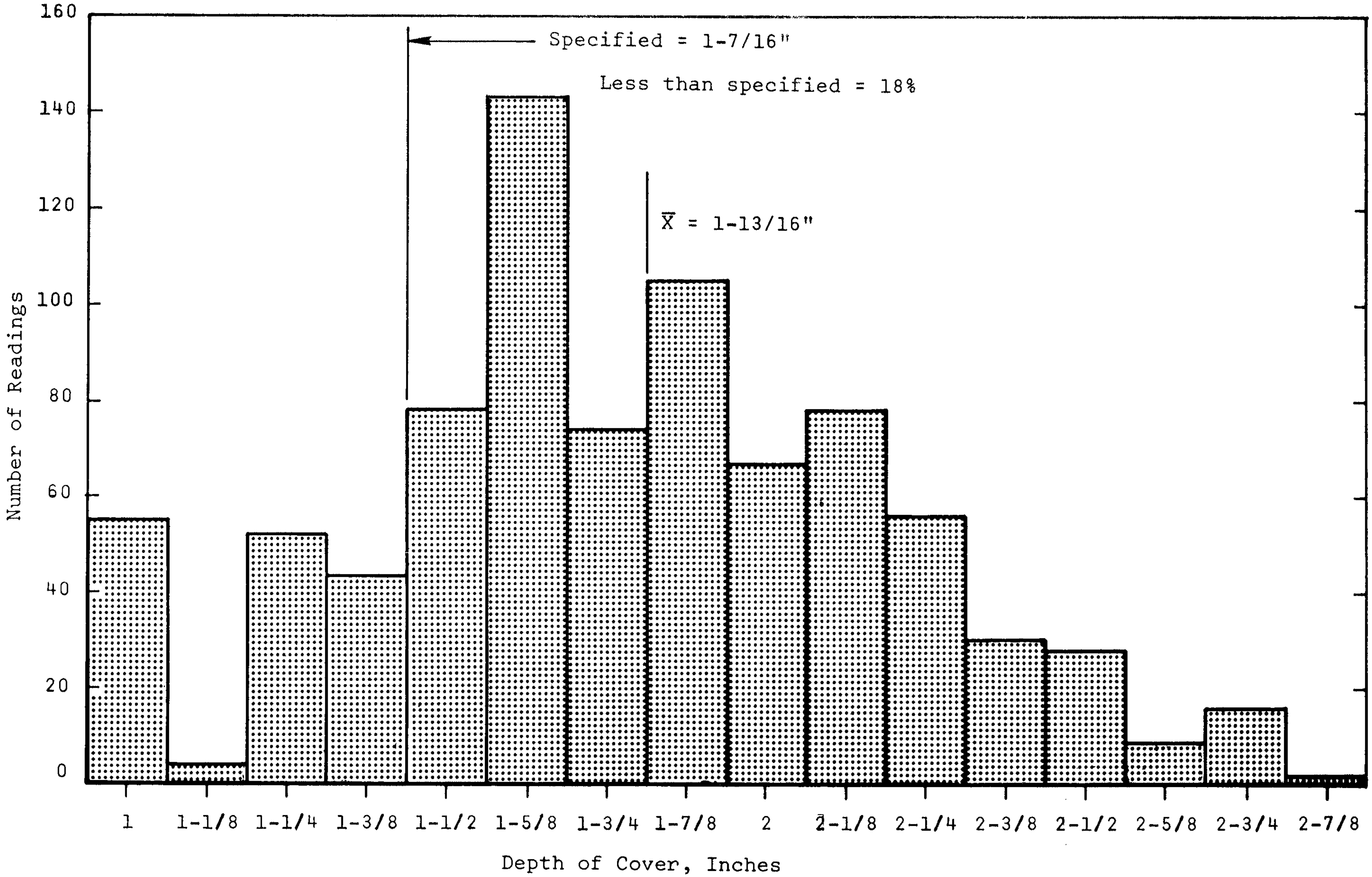


Figure 10. Distribution of cover depths — South Fork structure.

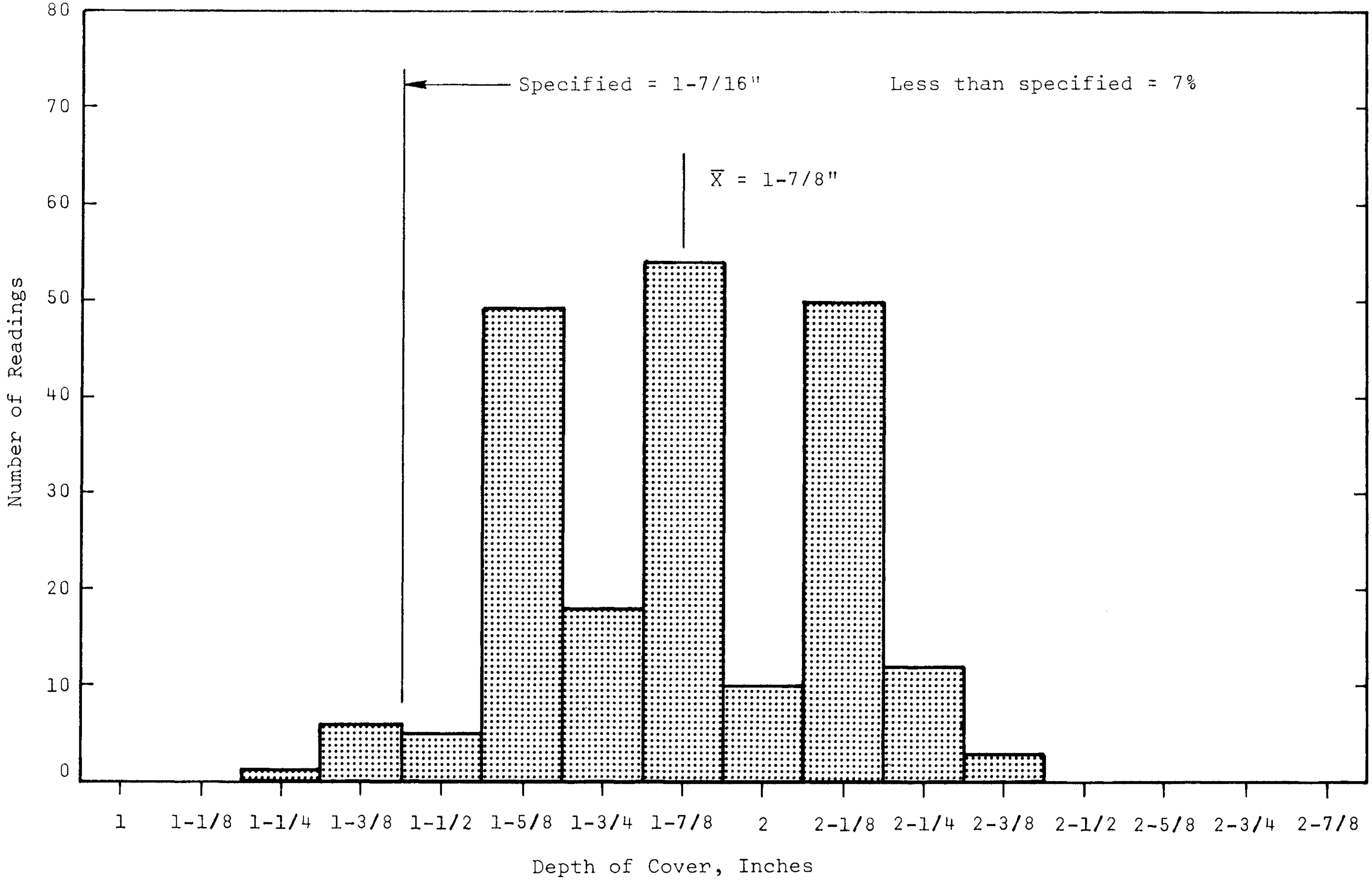


Figure 11. Distribution of cover depths — North Fork structure.

Electrical Corrosion Potentials

The electrical corrosion potentials were determined on a 5-ft. (1.5 m) grid on each of the experimental sections. A view of these measurements in progress is shown in Figure 12. The procedure was the same as that described in earlier Council reports (Smith 1973; Newlon 1974). Ground connections were made to steel armored expansion joints. It is now known that it is preferable to use the reinforcement as the ground connection. Cumulative frequency diagrams for the decks representing all spans, all experimental spans, and spans incorporating each of the study variables are presented in Figures 13-20. Electrical potentials greater than -0.35 volt with respect to the copper sulfate half-cell are generally considered to be indicative of active corrosion and potential differences below -0.20 indicate no corrosion. Potentials between -0.20 and -0.35 volt represent an uncertain corrosion condition (Clear and Hay 1973). The general shapes of all of the cumulative frequency diagrams are the same. There is no difference between the results from the experimental spans and those from all spans. The proportion of values less than -0.35 volt for each of the conditions is shown in Table 6.

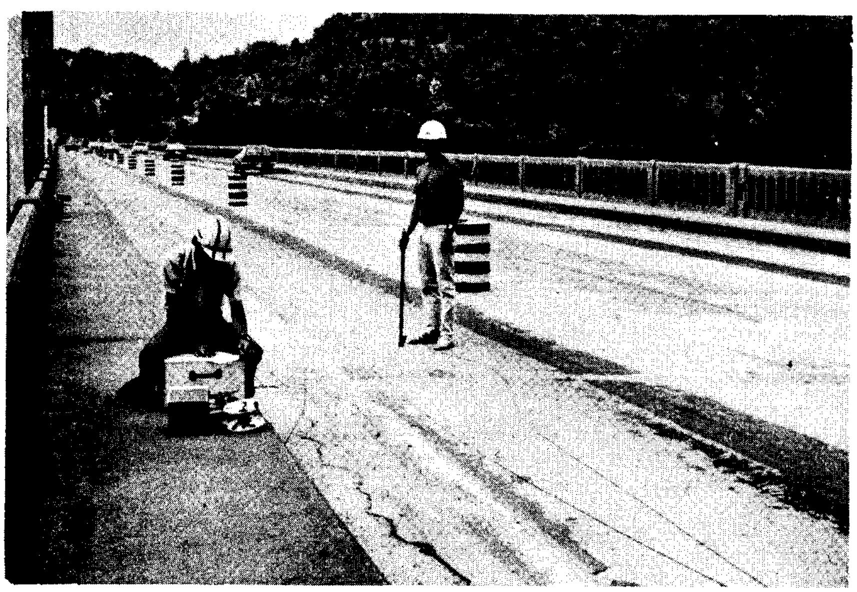


Figure 12. Corrosion potential measurements in progress.

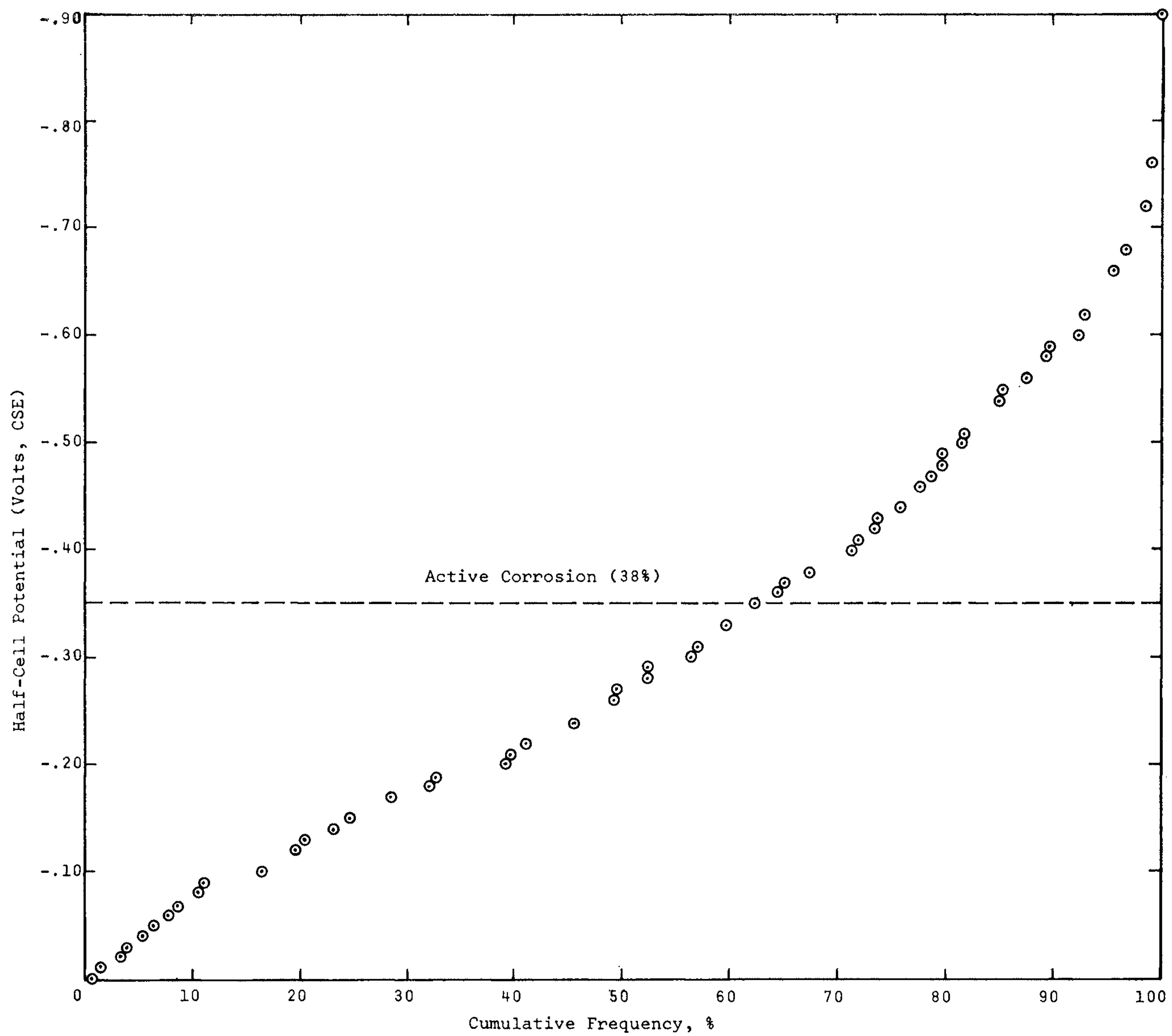


Figure 13. Corrosion potentials — all spans.

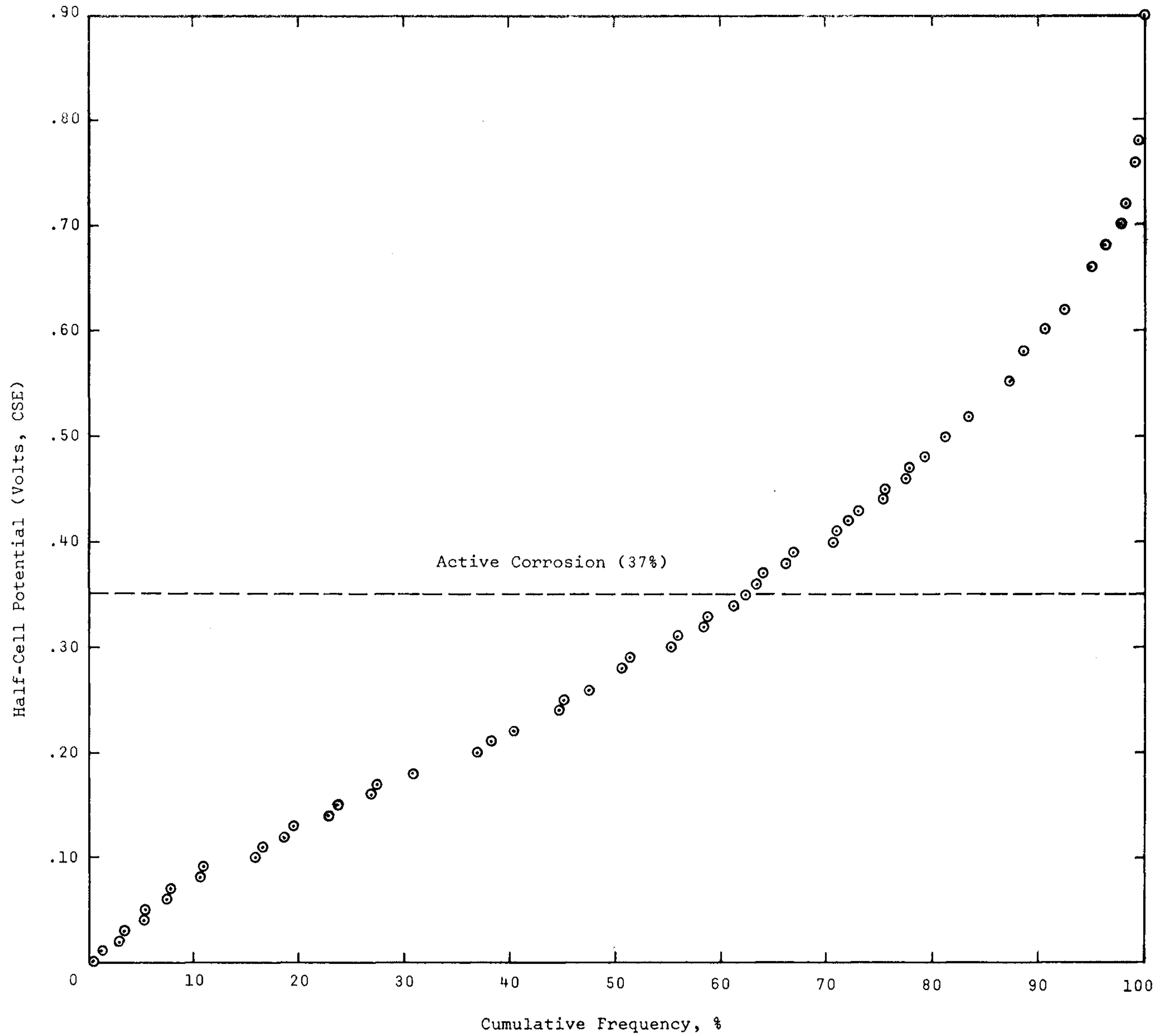


Figure 14. Corrosion potentials — all experimental spans.

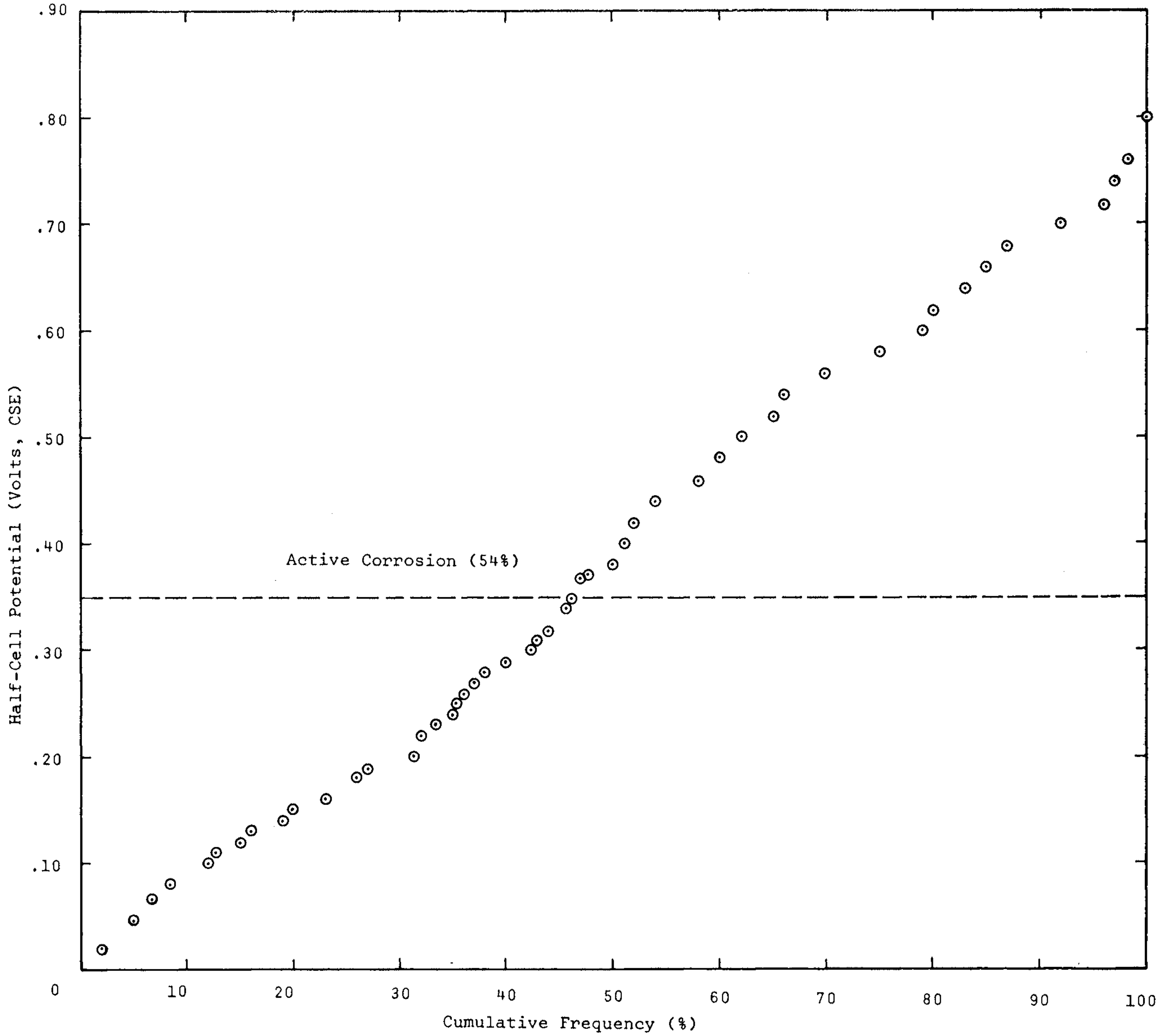


Figure 15. Corrosion potentials — Class A concrete — no lime added.

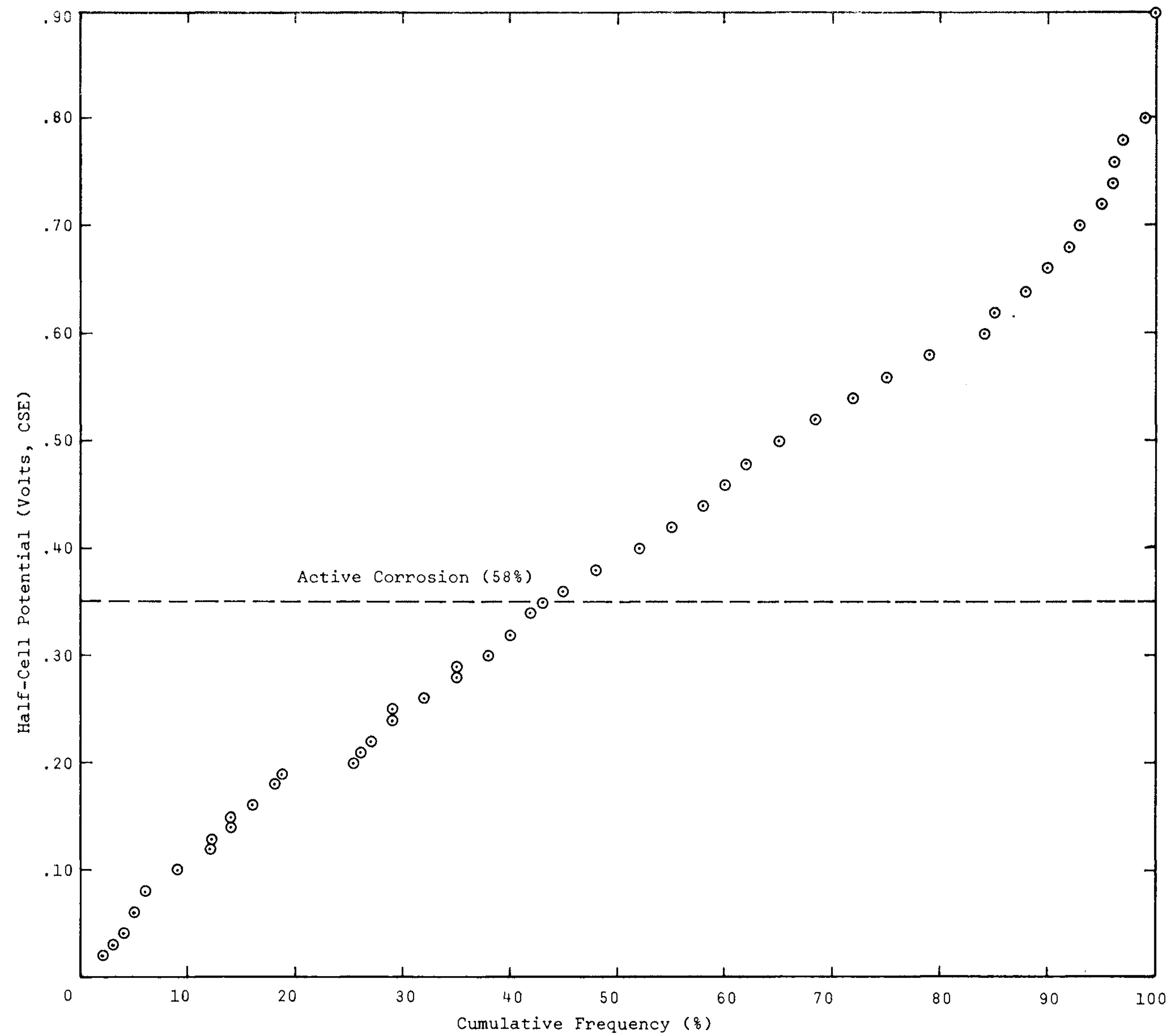


Figure 16. Corrosion potentials — Class A-4A concrete — 4% lime by volume.

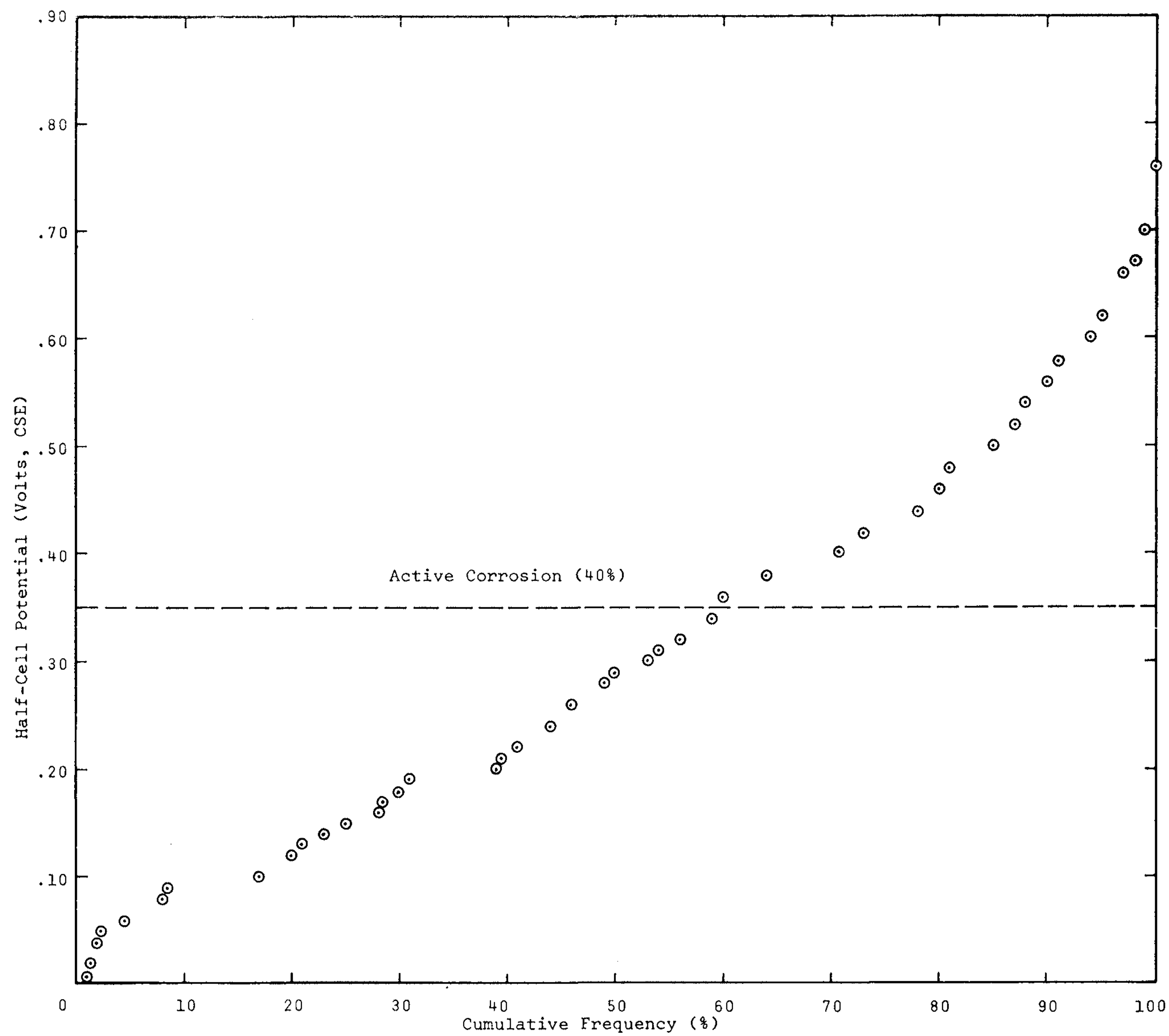


Figure 17. Corrosion potentials — Class A-4 concrete — 4% lime by weight.

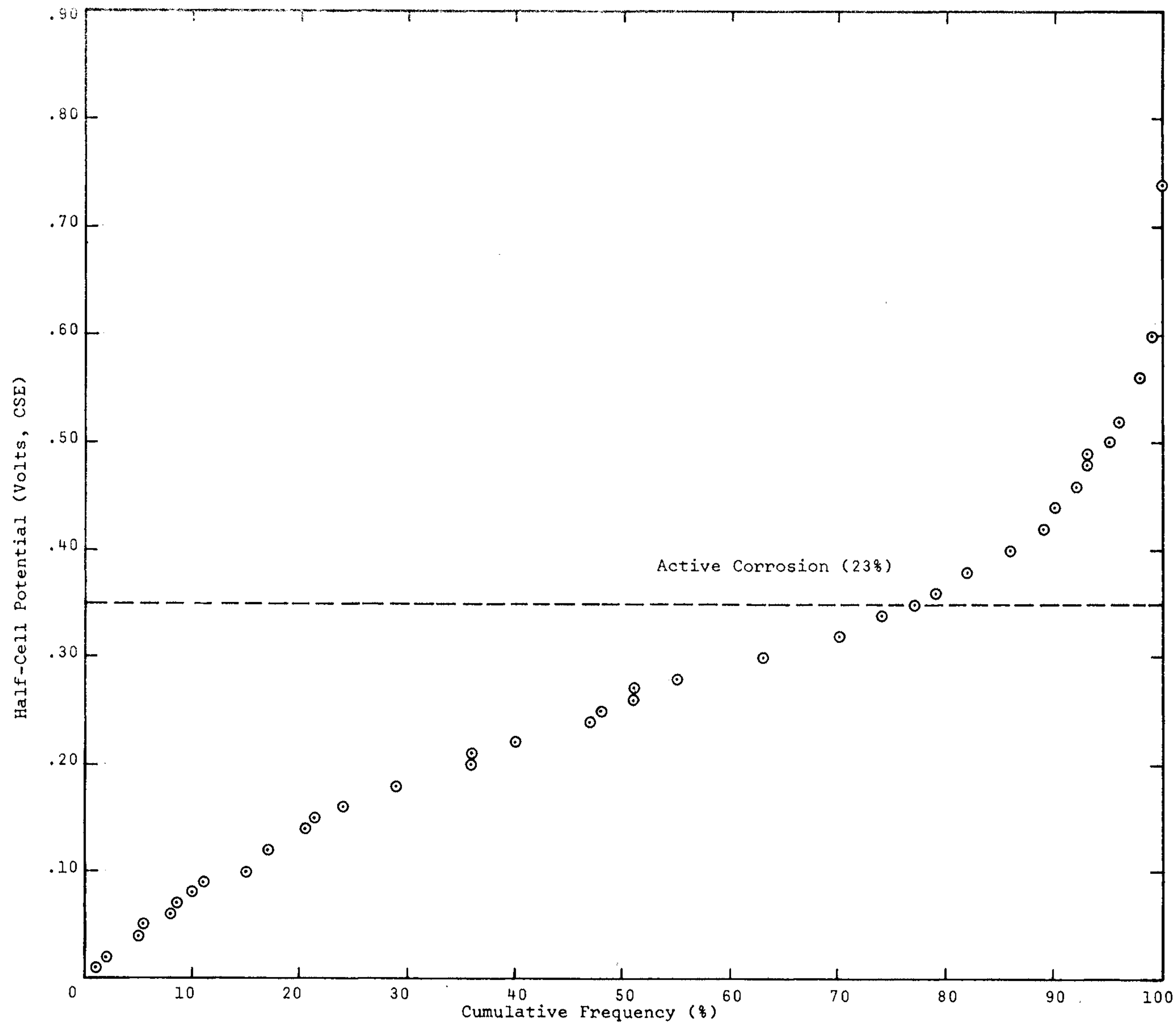


Figure 18. Corrosion potentials — Class A-8A concrete — 8% lime by volume.

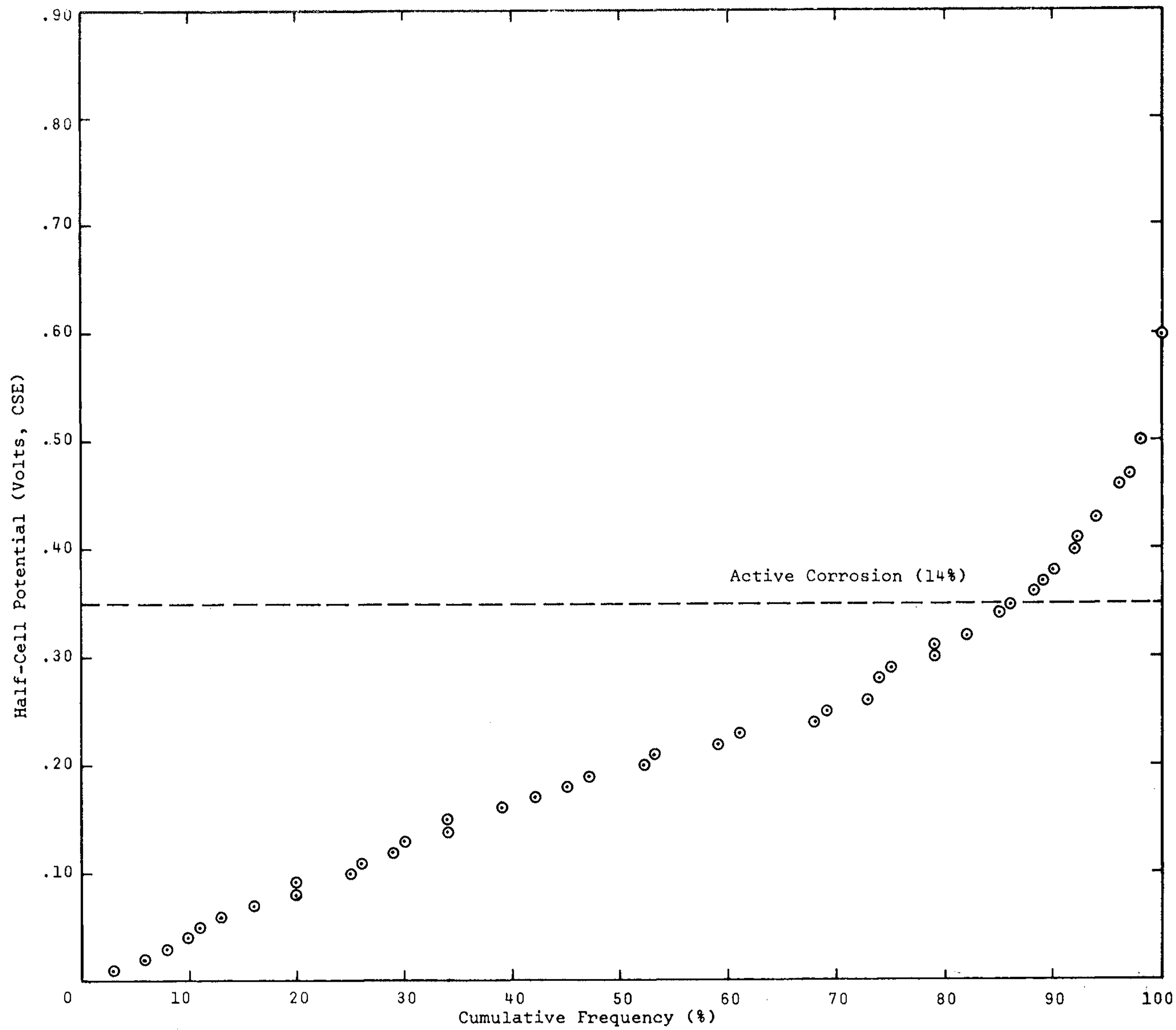


Figure 19. Corrosion potentials — Class A-8 concrete — 8% lime by weight.

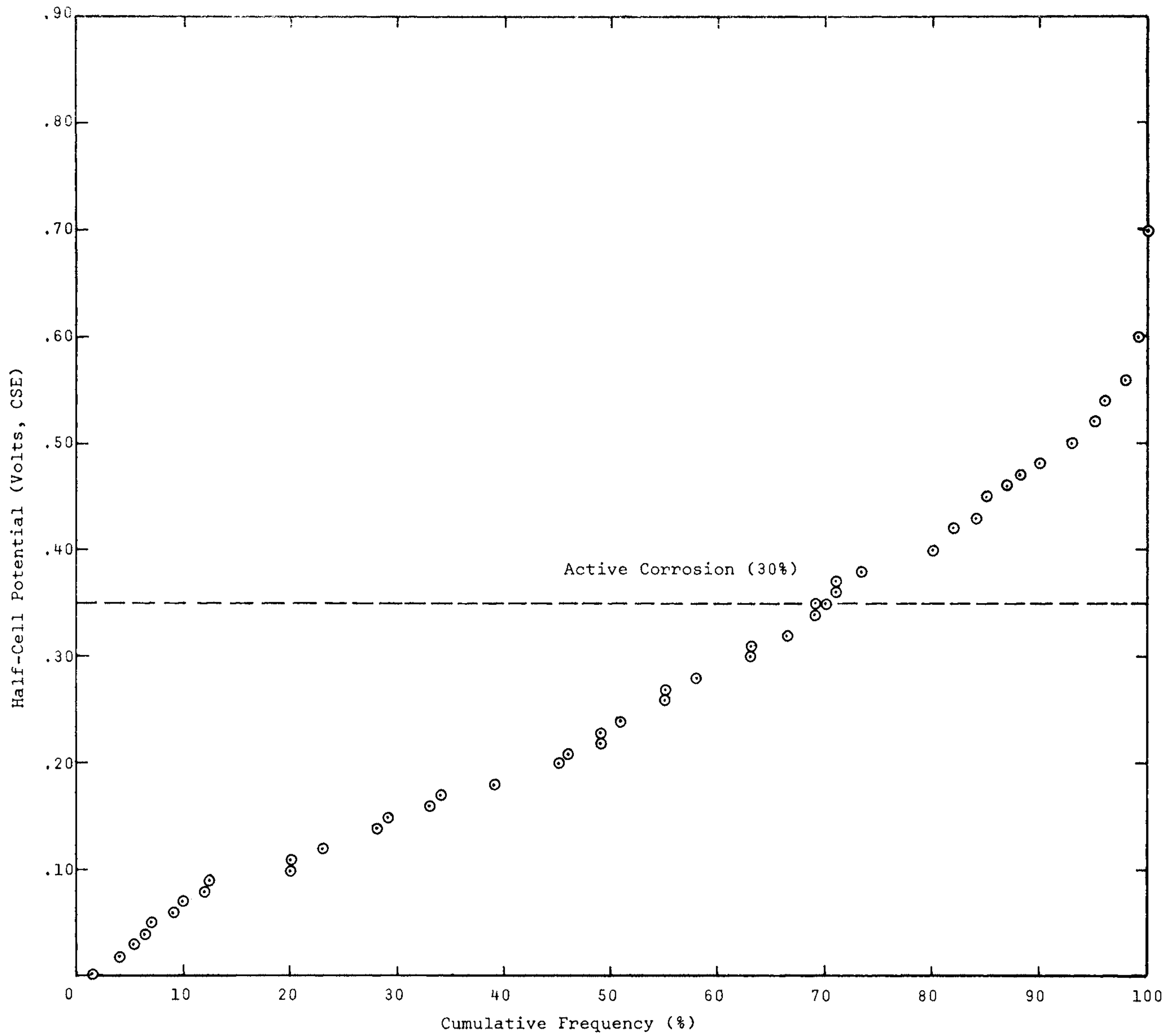


Figure 20. Corrosion potentials — Class A-12 concrete — 12% lime by weight.

Table 6

Proportion of Electrical Potential Measurements Below -0.35 volt with Respect to Copper Sulfate Half Cell

Condition	Weight of Lime Per 94 pounds of Portland Cement*	Percentage of Readings Less than -0.35 volt
All values (experimental and non-experimental)	varies	62
All experimental sections	varies	63
A	0	46
A-4-A	3.1	42
A-4	3.3	60
A-8-A	6.2	77
A-8	6.8	86
A-12	10.6	70

*1 pound-mass = 0.4536 kilogram

From the data developed in this project it appears that there is an improvement in corrosion resistance with increasing amounts of lime until the highest lime content is reached. In view of the age of the bridges, the fact that in the best case almost 90% of the readings are below -0.35 volt represents excellent protection of the reinforcement, particularly when the comparatively shallow concrete cover is considered.

An interpretation of these results is given later in the report following presentation of the results from the petrographic and chemical analyses.

Chloride Penetration

From each core, powder samples were taken for chloride analyses using the specific ion electrode titration procedure developed by Berman (1972). The Berman procedure has been modified by Clemeña, Reynolds, and McCormick (1976). The average chloride contents at a level approximately 1 in. (25 mm) below the surface are shown in Table 7.

Table 7

Chloride Contents 1" Below the Deck Surface

Condition	Weight of Lime Per 94 lb. of Portland Cement	Chloride Content % by Weight of Concrete Powder	Chloride Content by Weight of Concrete lb./yd.^{3}
A	0	0.119	4.71
A-4-A	3.1	0.130	5.14
A-4	3.3	0.133	5.26
A-8-A	6.2	0.113	4.47
A-8	6.8	0.126	4.99
A-12	10.6	0.128	5.06

Note: 1" = 25 mm
pound-mass = 0.4536 kilogram
pound-mass/cubic foot = 0.5933 kilogram/cubic meter

As seen in Table 7, there is no significant difference of chloride contents among the concretes containing various amounts of lime. This fact suggests that the protection against corrosion, if any, is probably not related to differences in permeability, because the average chloride levels are approximately the same. The protection, if any, must result from the lime mitigating the initiation or progress of the corrosion in the presence of chloride.

Current guidelines suggest that corrective action is necessary when the chloride content by weight of concrete exceeds 2.0 lb./yd.^{3} (1.2 kg/m^3) (Clear and Hay 1973). It is generally believed that the value should be 1.0 lb./yd.^{3} (0.6 kg/m^3). In any event, the chloride contents are all significantly higher than either threshold value, and the conditions necessary for corrosion obviously are present.

Petrographic and X-Ray Diffraction Analyses*

From the 69 cores obtained from the deck, 32 were selected for preliminary petrographic analysis by X-ray diffraction. Conditions represented by the cores are indicated in Table 8.

Table 8

Cores Selected for Analyses

Condition	Cores
A	S1B, S8A, N1C, N10A, 10-22*
A-4-A	S6A, S12D*, N8C, N17C*
A-4	S3A, S9D, N2C, N14A, N14C*
A-8-A	S7D, S13A(1)*, S13A(2), N9A, N9C, N13A*, N13C
A-8	S4A*, S4B, S10A*, N5A, N15A
A-12	S5A, S11B, N6C, N16C*, 6-6*
nonexperimental	N3A

*Exhibited spalling defect

The sections of each selected core were examined with a low power stereoscopic microscope. The concrete was seen to be non-air entrained. A moderate amount of irregular voids, generally classified as water voids, were seen in the cores, often trapped against aggregate particles. The voids were not of sufficient size or quantity to indicate abnormally wet concrete.

The void contents were determined on 7 of the cores using the procedures outlined in ASTM C457. One of these cores (S-13-A(1)) was air entrained, and subsequently was found to be part of an air entrained repair placed in 1970. The average air content for the

*The petrographic and X-ray diffraction analyses were conducted by or under the supervision of H. N. Walker. B. N. Marshall was directly responsible for the X-ray analyses.

remaining 6 cores was 2.17%, of which 0.65% had diameters less than 1 mm, and the remaining 1.62% were larger than 1 mm and obviously were what would generally be characterized as "water voids".

A summary of the petrographic examination is given in Appendix D.

X-ray Analyses

A small portion of each core was crushed gently in an iron container and several small fragments of mortar were selected which were free of coarse aggregate. The small mortar fragments were then powdered with an agate mortar and pestle. The powder obtained was mounted on a glass slide in a Phillips XR6-2500 and diffractograms from 5^{o} to 45^{o} were obtained for each sample.

As expected, all patterns showed the presence of quartz, portlandite, minor amounts of mica, gypsum, and feldspars. No calcium oxide was detected. Despite the fact that the examination of the thin sections showed these concretes to be well carbonated, only minor amounts of calcite were detected. No aragonite or vaterite were detected.

An attempt was made to correlate the amount of portlandite (calcium hydroxide) with the amount of hydraulic lime which had been added to the mixture. No correlations were found. It is conceivable that this lack of correlation may be due to errors in the sample preparation. However, because of the solubility of calcium hydroxide, it is probable that the amount of portlandite present in this concrete is due more to the permeability of the individual unit and the amount of water which has gone through it than to the amount of hydrated lime originally in the mixture.

DISCUSSION

Based upon the results that have been presented, the condition of the concrete in the bridge decks is summarized as follows:

1. The quality of the concrete as a material is excellent, considering its age, exposure to deicing chemicals and absence of air entrainment.

2. The chloride contents are approximately the same for all the types of concrete and are all well above levels necessary for corrosion.

3. Corrosion is occurring in most of the sections, but the potential for corrosion as indicated by electrical measurements varies inversely with the amount of hydraulic lime in the concrete.

4. Quality control on the project was excellent and probably better than on most projects constructed during the period.

The corrosion of reinforcement in concrete subjected to chlorides results from an electrochemical process in which current flow is established with anodic and cathodic areas developing along the bar. Material is lost at the anode. These areas can develop adjacent to each other and account for the behavior that was illustrated in Figure 8.

Even though the conditions for electrochemical corrosion might exist because of nonuniformity of the steel or differential concentrations of chlorides, corrosion is normally prevented by a "passivating" iron oxide film which rapidly forms on the steel surface in the presence of moisture, oxygen, and the water-soluble alkaline products formed during the hydration of the cement. The principal soluble product is calcium hydroxide, and the initial alkalinity of the concrete is at least that of saturated lime water (pH of about 12.4, depending upon the temperature). In addition, the seemingly relatively small amounts of sodium and potassium oxides in the cement further increase the initial alkalinity of the concrete or paste extracts and pH values of 13.2 and higher have been reported (Verbeck 1975).

No chemical analyses of the portland cement or hydraulic lime are available, but since the construction predated concern for alkali aggregate reactions, the alkalies (Na and K compounds) were probably above 0.6%. The hydraulic lime probably also contributed additional amounts of these compounds. Although any explanation is speculative in the absence of specific data, if the relationship between reduced corrosion potential and increased additions of hydraulic lime is real, the explanation must lie in the contribution of calcium hydroxide by the hydraulic lime to that generated by the hydration of dicalcium silicate and tricalcium silicate to form a "reservoir" of calcium hydroxide to keep the pH high for a longer period of time than would be true in the absence of the added lime. In addition, if significant amounts of sodium and potassium were contributed by the hydraulic lime, they would also increase the pH, which in turn would passivate the steel. While no analyses of the cement or lime are available, there is some indirect confirmation that the total alkalies of such a mixture were high, based on a relatively rare documented but unpublished occurrence of alkali-silica reaction in a bridge built at approximately the same time using a combination of portland cement and the same hydraulic lime.

The performance of corrosion-inhibiting admixtures has been varied but generally unsatisfactory (Griffin 1975). Studies by Berman (1975) however, suggest the desirability of investigating the addition of alkaline materials to concrete containing or exposed to chloride ions.

It is likely that the major influence on the generally excellent performance of all concrete on the Shenandoah project resulted from the increased inspection and testing exercised as described in Appendix B.

CONCLUSIONS

Based upon the results and interpretations in this report, the following conclusions appear warranted.

1. The performance of the concrete in the decks has been excellent considering the age of the structure and the lack of air entrainment. There is no significant scaling or other general deterioration.

2. The major defect is surface spalling from corrosion of the reinforcement. This is not surprising in view of the high chloride levels and comparatively shallow concrete cover above the top reinforcement.

3. Based upon electrical potential measurements, the corrosion potential decreases with increasing additions of hydraulic lime, for additions of 4% and 8%, and increases slightly between 8% and 12%. If this reduction is real, it must be related to some initial passivation of the reinforcement from either the lime itself or included alkalies that over the years was overcome but which still is reflected in the extent of corrosion.

4. The overall excellent durability of the concrete in these decks as compared with the concrete in contemporary decks undoubtedly reflects the greater than average testing and inspection exercised in connection with the experimental features of the project.

5. Within the limits of the chemical and petrographic analyses, no unusual reaction products or characteristics were observed among the several types of concrete used.

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ACKNOWLEDGMENTS

H. N. Walker and John Reynolds conducted the analyses of the hardened concrete. The field observations were made by Clyde Giannini, Lewis Woodson, and Bobby Marshall. Bobby Marshall also conducted the X-ray and void content analyses. Celik Ozyildirim contributed in many ways, including assisting with the field measurements and analyzing much of the data from these observations. Appreciation is expressed to all of these people for their efforts.

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APPENDIX A — COPY

VIRGINIA DEPARTMENT OF HIGHWAYS
Richmond, Virginia.

July, 1940.

SPECIAL PROVISIONS TO GOVERN EXPERIMENTAL FEATURES IN CONNECTION WITH THE CONSTRUCTION OF BRIDGES AT RIVERTON, VIRGINIA - Proj. 1015-S1,2.

The following special provisions are included in order to provide for a study of the effect of using hydraulic hydrated lime on the workability of the concrete used in these structures. Mixtures containing various percentages of hydraulic hydrated lime will be used in certain of the piers and in certain sections of the deck, as shown on the plans.

The following special provisions are to be included in the specifications:

MATERIALS

Portland Cement: Suitable arrangements shall be made by the contractor to insure that all of the portland cement used in the concrete in each bridge shall come from a single bin.

Hydraulic hydrated lime: The hydraulic hydrated lime shall comply with the requirements for high calcium hydraulic hydrated lime of the Tentative Specifications for Hydraulic Hydrated Lime for Structural Purposes, Designation C 141-38T, of the American Society for Testing Materials.

Fine aggregate: Section 203.03 of the standard specifications shall be modified by adding the following:

> The above gradation represents the extreme limits which shall determine suitability for use from all sources of supply. The gradation from any one source shall be reasonably uniform and not subject to the extreme percentages of gradation specified above. For the purpose of determining the degree of uniformity, a fineness modulus determination shall be made upon representative samples, submitted by the contractor, from such sources as he proposes to use. Fine aggregate from any one source having a variation in fineness modulus greater than 0.20 either way from the fineness modulus of the representative sample submitted by the contractor may be rejected. Fine aggregate from different sources of supply shall not be mixed or stored in the same pile nor used alternatively in the same class of construction or mix, without permission from the engineer.
>
> (The fineness modulus of an aggregate is determined by adding the percentages by weight retained on the following sieves having square openings, and dividing by 100: 3 in., $1\frac{1}{2}$ in., 3/4 in., no. 4, no. 8, no. 16, no. 20, no. 50, no. 100.).

PROPORTIONS

Section 336.05 of the standard specifications shall be modified by adding the following:

Six classes of concrete shall be used in the experimental work, as follows:

Class A. Standard class A concrete as specified in section 336.05.

Class A-4. Class A concrete in which 4 percent of the cement by weight will be replaced by an equal absolute volume of hydraulic lime.

Class A-3. Class A concrete in which 8 percent of the cement by weight will be replaced by an equal absolute volume of hydraulic lime.

Class A-12. Class A concrete in which 12 percent of the cement by weight will be replaced by an equal absolute volume of hydraulic lime.

Class A-4A. Class A concrete to which hydraulic lime will be added in an amount equal to 4 percent of the absolute volume of the cement.

Class A-8A. Class A concrete to which hydraulic lime will be added in an amount equal to 8 percent of the absolute volume of the cement.

The weights of solid materials in pounds and the maximum allowable volume of water in gallons per sack (94 pounds) of portland cement shall be as follows:

Quantities of materials per sack (94 pounds) of portion cement						Approximate volume of concrete sack (94 lb.) of cement (Note).
Class	Cement	Lime	Water (maximum)	Weight of aggregate Fine	Coarse	
	Pounds	Pounds	Gallons	Pounds	Pounds	Cubic feet
A	94	-	6.00	200	301	4.31
A-4	94	3.3	6.21	208	314	4.48
A-8	94	6.8	6.43	217	327	4.67
A-12	94	10.6	6.68	227	342	4.88
A-4A	94	3.1	6.20	200	301	4.36
A-8A	94	6.2	6.40	200	301	4.40

Note: These quantities are based on the use of the maximum allowable water content. In general the yield of concrete per sack of cement may be expected to be somewhat lower than shown, due to the use of less than the maximum allowable content.

TESTS

Modify section 336.07 by the addition of the following:

In addition to the test specimens normally required under this section, the contractor shall furnish, without extra charge, sufficient concrete to permit the fabrication of test specimens in accordance with the following schedule:

Piers: In general, three 6 in. by 12 in. cylinders will be cast for each lift in each of the piers designated as experimental.

Deck: In general, three 3 in. by 12 in. cylinders and three 6 in. by 6 in. by 36 in. beams will be cast for each continuous placement in each section of the deck designated as experimental.

FIELD LABORATORY

The contractor shall provide, without extra charge, a suitable building to be used as a field laboratory and office by the engineers in charge of the experimental work on this project. It shall have a floor area of not less than 150 square feet, with at least one window and a door with locks and shall be furnished with a suitable table or shelf. The windows shall be so arranged as to permit ventilation.

VIRGINIA DEPARTMENT OF HIGHWAYS
Richmond, Virginia.

Proposed Bridge over South Fork - Shenandoah River
Project 1015 - B1

Location of Various Classes of Concrete

PIERS

Pier Number	Class of Concrete	Character of Construction
4	A	Experimental
3	A-4	"
6	A-8	"
7	A-12	"
8	A-4A	"
9	A-8A	"
1-2-5-10-11	A	Nonexperimental

FLOOR SLAB

Section Number	Class of Concrete	Character of Construction
1 and 8	A	Experimental
3 and 9	A-4	"
4 and 10	A-8	"
5 and 11	A-12	"
6 and 12	A-4A	"
7 and 13	A-8A	"
2-14-15-16-17-18-19	A	Nonexperimental

Proposed Bridge over North Fork - Shenandoah River
Project 1015 - B

Location of Various Classes of Concrete

PIERS

Pier Number	Class of Concrete	Character of Construction
3	A	Experimental
2	A-4	"
6	A-8	"
7	A-12	"
8	A-4A	"
9	A-8A	"
1-4-5	A	Nonexperimental

FLOOR SLAB

Section Number	Class of Concrete	Character of Construction
1 and 10	A	Experimental
2 and 14	A-4	"
5 and 15	A-8	"
6 and 16	A-12	"
8 and 17	A-4A	"
9 and 13	A-8A	"
3-4-7-11-12	A	"

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APPENDIX B — COPY

RECORDS

The following records were kept on the experimental concreting on the Riverton Bridges. There are eleven (11) books.

Book A-1, a loose-leaf folder, contains:

1. The special provisions about one experimental concrete.
2. A sheet showing the experimental mixtures and explaining them.
3. A sheet showing the standard weights for the experimental mixtures.
4. Picture sketches of the North and South Fork Bridges, showing the location of the various classes of concrete.

Also contained in this book are data sheets for concrete proportioning that were made up for each pour of the superstructures on the three bridges, the North and South Fork Bridges and the Crooked Run Bridge. The Public Roads Administration received this form for each pour of the substructure and superstructure, but copies retained by the Department were on the superstructure only. The variation in the procedure of keeping these records, that is, retaining the copies of these sheets, was due to changing the personal organization of the Department's representative on the job. This type of sheet is concise and complete and is recommended for use by our engineers.

Book A-2, entitled "Material Tested Record on Concrete Ingredients", contains tested quantities of the cement, hydraulic hydrated lime, sand, crushed stone and water. In the grand summary of this book are shown the total quantities of the different classes of concrete and the ingredients that went into this concrete. This material book covers all three of the bridges.

Books A-3, A-4 and A-5 are entitled "Concrete Pouring Record", and A-6 "Material Used Record" on concrete ingredients and refers to the concrete work on the South Fork Bridge.

The North Fork pouring records are numbered A-7 and A-8 and the "Material Used Record" on the concrete ingredients on this bridge is No. A-9.

Books A-10 and A-11 are included in one volume entitled "Concrete Pouring Record and Materials Used Record" for the Crooked Run Bridge.

In the books entitled "Concrete Pouring Record" are shown the data for each day's pour and specimen records for these pours.

In the substructure the cylinders that were sent to the Public Roads Administration to be broken were numbered in consecutive order beginning with one (1). Those cylinders that were sent to our own laboratory for breaking were numbered V-1, V-2, etc.

The cylinders on the superstructure that were sent to the Public Roads Administration were numbered S-1, S-2, etc., and the cylinders that were sent to our own laboratory were numbered VS-1, VS-2, etc.

The beams made on the superstructure were numbered B-1, B-2, B-3, etc., and VB-1, VB-2, VB-3, etc. These beams were all broken in the field and one end from each beam was sent to the Public Roads Administration. Each beam was broken twice and the two breaks and the average break were recorded.

- ORGANIZATION -

Mr. H. F. Piercy, Senior Highway Engineer, was in charge of the construction of the Riverton Bridges. The laboratory work was taken care of by Mr. G. R. Kauffmann, Mr. J. W. Mitchell, and Mr. Percy E. Hood. Mr. Kauffmann left the Department January 31, 1941, and went into the Army. Mr. Mitchell resigned from the Department April 8, 1941, and Mr. Percy E. Hood was on the job from beginning to end.

While pouring the substructure there were three (3) men on the laboratory work alone. One of these men stayed at the batching plant where the field laboratory was located. His duties were —

- Checking weights on aggregates and bulk cement
- Running moisture tests
- Changing weights as required and running grading analyses on sand
- Making the cylinder specimens (Samples of the concrete that was being poured were sent by truck to this man)

Another man, who observed the work at the mixer, had the following duties:

- Checking the mixing time of the concrete
- Observing the water device and concrete mixer operator
- Checking by eye the moist and dry conditions of the aggregate from time to time
- Observing the concrete in the "Pump-Crete Hopper" before it ran into the "Pump-Crete pipe line
- Running consistency tests (slump)

The third man observed and supervised the work going on at the mixer, the field laboratory and where the concrete was being placed. His judgement was used on the workability of the concrete. He also worked up the pouring records.

The specimens were placed in the field laboratory curing room and cured at a continuous temperature of 70°. Practically all of these specimens were broken at twenty-eight (28) days.

While pouring the superstructure four men were used in the concrete work alone, the extra man being required in view of the fact that the volume of specimens made was so great.

The beams and cylinders were made at the mixer. The "Pump-Crete" machine with the pipe line was used for the superstructure concreting. One of the laboratory inspectors at the mixer observed the concrete in the "Pump-Crete Hopper" before it went into the pipe line. Cement from cloth bags was used in the superstructure. This bag count was kept by the inspector at the mixer. The field laboratory inspector was running moisture tests continually, the number of tests depending on the length of the pour. He sent the results of these tests to the inspector supervising the general concrete work, who changed the weights as required and made up the pouring record during the day's pour.

The volume of concrete laboratory work kept at least two men busy most of the time even when we were not actually pouring concrete. Beams that had reached the required age had to be broken, data had to be worked up and recorded, and specimen molds had to be cleaned. The beams and cylinders were cured outside, being buried in sand piles near the point of pour and practically all of these specimens were broken in twenty-eight (28) days. The superstructure was poured during warm weather.

(It is suggested here that for ordinary concrete jobs three men be used to insure good control of concreting, one of these men being placed at the batching plant running moisture tests, etc., one looking after the placement of the concrete.)

— CONCRETE CONTROL —

Conclusions and suggestions for this control drawn from the experimental concrete work at Riverton.

The control of concrete for any job begins in the laboratory where the ingredients of concrete are characterized and proportioned. Our specifications for concrete and concrete ingredients are the result of laboratory work. To a certain practical degree our present specifications will give us good uniform concrete in relation to strength and workability if carried out by engineers on concrete jobs. Consciousness of this fact is most important. The uniform method of proportioning ingredients on all the concrete jobs throughout the State must be carried out in order to bring about more uniform strengths.

The ingredients of a given class of concrete on a given job should be proportioned uniformly throughout the length of the job. At Riverton the fine aggregate and the coarse aggregate were proportioned according to the grading analyses that were being made constantly by the plant inspector. Every effort was made by the plant inspector to insure uniformity in both the coarse aggregate and the fine aggregate gradings, and as a result the ratio of fine to coarse remained consistent throughout the job. The uniformity of cement in relation to the aggregate is, of course, constant in a given class of concrete; therefore, the only ingredient of the concrete to be varied in quantity from time to time is water.

Of course, the type of aggregate throughout the State is not all uniform; that is, on some jobs gravel is used, on others crushed stone is used. This, of course, will result in a difference in the amount of water required for a given workability (slump). Where crushed stone is used more water may be required than the maximum shown in our specifications. This was particularly true at Riverton where we had to use in regular Class A concrete water-cement ratios averaging around .85 (3" to 5" slump). At the present time the Class A concrete being used in the Falmouth Bridges is requiring a water-cement ratio of around .76 for the same slump. This difference in aggregates and the water cement ratios required on the two types of aggregates should not result in great differences of strength if everything else, that is, the proportions of ingredients due to the grading of the aggregates, is kept consistent on a given job. It has been found that crushed stone in most cases gives a higher strength concrete than gravel although concrete made from crushed stone requires more water.

A typical design of a Class A mix used at Riverton follows.

Cubic Foot Solid Proportions shown —

Cement	0.48	Cu.	Ft.	Solid
Water	0.87	"	"	"
F.A.	1.35	"	"	"
C.A.	1.69	"	"	"
Yield	4.39	"	"	"

Cement Factor = 6.15 bags per Cubic Yard

Throughout the job the cement, fine aggregate and coarse aggregate remained consistent as shown above but the water varied at times. The reasons for this variation are due to the atmospheric conditions and type of structure being poured. This variation in water on any given job is to be accepted as a fact, but if the engineer will make the effort to know that he is actually getting, for instance, a water-cement ratio of .85 in a given class of concrete, his control of the concrete throughout the job should be consistent and uniform.

To actually know that one is using a water-cement ratio of, for instance, .85, much work is involved. This work begins at the batching plant where moisture tests must be run constantly. A man should be placed there throughout the length of the pour, running these tests and adjusting the weight of the aggregates whenever his moisture test results call for these changes. In general our specifications do not require that aggregate be stored for a certain length of time before being used so the only way to know just how much water is in the aggregate is to run moisture tests. At times at Riverton we were using sand from both stockpile and new carloads during the day's pour. We were also using stone from the stockpile and absolutely dry stone that was being hauled from the quarry near the job. This variation in moist and dry material called for many moisture tests but during the day's pour we knew to a practical degree how much free water we had in our aggregates.

(It is suggested here that where aggregate is being hauled from the plant for a long distance to the mixer that these moisture tests be run not at the plant but at the point of pour on the job. This is especially necessary where sand and gravel is being loaded into batching trucks shortly after leaving washing plants and newly loaded barges. The reason for this is that much free water in the aggregates is lost during long hauls and that moisture tests run at the plant would not hold true on that same material after it had arrived at the job.)

The next factor to consider is the concrete mixer being used. The mixer should be calibrated before being used and at different intervals throughout the length of the job. There should be no question in the engineer's mind as to the accuracy of the water gauge. The efforts made in controlling the free water in the aggregates by moisture tests will be of no value if the water device on the mixer is not in good serviceable condition. This is of utmost importance because wet and dry batches result not only in inconsistency of workability but in strengths. The timing of the mixer should be set up at the very beginning and held constant throughout the length of the job. Knowing the exact amount of free water and knowing the exact amount of water being added results in uniform batches of concrete. Specimens made over a period of time that are drawn from uniform batches of concrete result in uniform strengths. Cores drilled from concrete that was poured in uniform batches should result in uniform strengths. Therefore, the results in workability and uniform strength of concrete depend to a large degree upon the efforts made by the engineer to keep consistent at all times the proportion of the ingredients being used to make up the concrete.

- CONCLUSIONS -

This experiment was made for the purpose of studying the effect of using hydraulic hydrated lime on the workability of the concrete used in the Riverton Bridges.

The Public Roads Administration had an engineer on the job throughout the pouring of the experimental concrete and the Administration will no doubt draw its own conclusions as to the workability of the various classes of concrete.

No clear-out decision can be made by our laboratory until charts and curves, etc., have been made from the data recorded on the job. It is, however, interesting to note at this time that the uniform strengths of the specimens made on the job are a good result.

Signed Percy E. Hood
Percy E. Hood

APPENDIX C — COPY

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION
Washington, D. C.

REPORT ON SAMPLE OF 19 CONCRETE CYLINDERS

Laboratory No. 52853 to 52862 incl. Date March 15, 1941.
Name
Identification marks See Below
Submitted by Shreve Clark, Engineer of Tests, Department of Highways, Richmond, Va.
Sampled Jan. 20, 21, 23, 25, 28, 30, 31, Received February 4, 1941.
Sampled from February 1, 1941.
Quantity represented
Sources of material
Location used or to be used South Fork Bridge and North Fork Bridge, Riverton, Va.
Examined for

TEST RESULTS

Laboratory number	Identification number	Crushing strength, lb. per sq. in. Age 28 days
52853	131	5,660
	132	6,130
52854	133	5,470
52855	134	5,840
	135	5,550
52856	136	5,340
52857	137	5,730
52858	138	5,030
	139	4,970
	140	7,670
52859	141	5,510
	142	6,600
	143e	5,520
52860	144	6,130
	145	4,490
52861	146	5,640
	147	5,280
	148	5,020
52862	149	5,710

Specimens stored in moist air while in concrete laboratory

E. F. Kelley,
Chief, Division of Tests

Per W. E. G.

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION
Washington, D. C.

REPORT ON SAMPLE OF 24 CONCRETE CYLINDERS

Laboratory No. 53131 to 53138 incl. Date April 30, 1941.
Name
Identification marks See below
Submitted by Shreve Clark, Engineer of Tests, Department of Highways, Richmond, Va.
Sampled March 10, 13, 14, 19, 20, 22, 26, 27, 1941. Received April 3, 1941
Sampled from
Quantity represented
Sources of material
Location used or to be used North Fork Bridge, Riverton, Va.
Examined for

TEST RESULTS

Laboratory number	Identification number	Crushing strength, lb. per sq. in. Age 28 days
53131	215	4,940
	216	4,890
	217	5,210
53132	218	4,390
	219	4,640
	220	5,810
53133	221	5,230
	222	5,610
	223	4,680
53134	224	5,040
	225	5,310
	226	5,370
53135	227	5,870
	228	5,490
	229	5,770
53136	230	5,540
	231	5,490
	232	5,840
53137	230C	5,890
	231C	5,910
	232C	5,790
5138	233	5,350
	234	4,840
	235	5,030

Specimens stored in moist air while in concrete laboratory

E. F. Kelley
Chief, Division of Tests

Per W. E. G.

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION
Washington, D. C.

REPORT ON SAMPLE OF EIGHT CONCRETE CYLINDERS

Laboratory No. 53139, 53224, 53225 Date May 16, 1941.
Name
Identification marks See below
Submitted by Shreve Clark, Engineer of Tests, Department of Highways, Richmond, Va.
Sampled March 27, April 4, 11, 1941 Received April 3, 14, 1941.
Sampled from
Quantity represented
Sources of material
Location used or to be used Bridge, Riverton, Va.
Examined for

TEST RESULTS

Laboratory number	Identification number	Crushing strength, lb. per sq. in. Age 28 days
53139	236	5,260
	237	5,380
	238	5,150
53224	239	6,500
	240	6,480
	241	6,390
53225	V-97	5,000
	V-98	5,130

Specimens stored in moist air while in concrete laboratory.

E. F. Kelley
Chief, Division of Tests

Per W. S. G.

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION
Washington, D. C.

REPORT ON SAMPLE OF 18 Concrete Cylinders

Laboratory No. 54214 to 54219 incl. Date August 26, 1941
Name
Identification marks See below
Submitted by Shreve Clark, Testing Engr., Dept. of Hwys., Richmond, Va.
Sampled f July 16, 17 and 18, 1941
Sampled from
Quantity represented
Sources of material
Location used or to be used: South Fork Bridge, Riverton, Va.
Examinee for

TEST RESULTS

Laboratory number	Identification number	Crushing strength, lb. per sq. in. Age 28 days
54214	S-45	5,080
	S-46	5,530
	S-47	5,640
54215	S-48	5,760
	S-49	5,730
	S-50	5,330
54216	S-51	5,270
	S-52	5,400
	S-53	5,200
54217	S-54	5,290
	S-55	5,200
	S-56	5,280
54218	S-57	5,350
	S-58	4,980
	S-59	5,300
54219	S-60	4,080
	S-61	4,830
	S-62	4,820

Specimens stored in moist air while in concrete laboratory.

E. F. Kelley
Chief, Division of Tests

Per W. E. G.

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION
Washington, D. C.

REPORT ON SAMPLE OF 20 Concrete Cylinders

Laboratory Name 54207 to 54213 incl. Date August 26, 1941
Identification marks See below
Submitted by Shreve Clark, Testing Engr., Dept. of Hwys., Richmond, Va.
Sampled July 11, 12, 14 and 15, 1941
Sampled from
Quantity represented
Sources of material
Location used or to be used: South Fork Bridge, Riverton, Va.
Examined for

TEST RESULTS

Laboratory number	Identification number	Crushing strength, lb. per sq. in. Age 28 days
54207	S-25	5,110
	S-26	5,270
	S-27	5,480
54208	S-28	4,990
	S-29	5,230
54209	S-30	4,990
	S-31	5,020
	S-32	5,020
54210	S-33	5,040
	S-34	4,860
	S-35	5,190
54211	S-36	4,310
	S-37	4,270
	S-38	4,790
54212	S-39	5,100
	S-40	5,410
	S-41	5,930
54213	S-42	5,540
	S-43	5,490
	S-44	5,000

Specimens stored in moist air while in concrete laboratory

E. F. Kelley,
Chief, Division of Tests

Per W. E. G.

APPENDIX D

PETROGRAPHIC EXAMINATION OF SELECTED CORES OF EXPERIMENTAL CONCRETE, HYDRATED-LIME ADDITIVE, FROM FRONT ROYAL

by

Ms. H. N. Walker

Thirty cores were selected for petrographic examination. They were cut, and a finely ground surface was produced on each. The data obtained from the examination of these surfaces are summarized in the attached table.

Eight standard thin sections were produced from an assortment of cores representing all the conditions of fabrication. Each thin section blank was cut from the middle of the core about 1 in. (25 mm) above the bottom steel. The sections were examined at magnifications of up to 500x and with both plane and crossed polarized light.

It was not possible to detect any differences in the optical properties of the paste that could be attributed to the different amounts of hydrated lime that had been used in the fabrication of the concretes. Considering the depth at which the sections were cut, all of the thin sections showed a high degree of carbonation. Scattered minute crystals of calcium carbonate were found throughout the paste. Heavy carbonation was found as external rims on some of the coarse aggregate (dolomitic limestone). This carbonation is most intense when found between two fragments of limestone or in a reentrant angle in the limestone surface. It appears as though the limestone ($CaCO_3$) contained sufficient excess carbonate ions to cause the carbonation. Very heavy carbonation was also found around the hydrous iron oxide pellets occurring in the fine aggregate. These pellets are composed of the minerals goethite and limonite and, aside from their hydrous nature, it is difficult to theorize as to why they might induce carbonation of adjoining paste.

A portion of the dolomitic limestone aggregate is of the type that can cause deterioration with high alkali cement. The only alkali-carbonate reaction noted was the occasional formation of a dark rim. No peripheral, longitudinal, or random cracking of the aggregate that could be attributed to alkali carbonate reaction was observed.

A very minor portion of the fine aggregate was reactive chert, and a transparent gel rim could be seen on a few chert particles.

No alkali-silica deterioration was observed. No associated cracking was seen. Occasionally, a thin layer of silica gel could be found lining a void, but no filled voids were found.

The results obtained with the X-ray diffraction must be considered in light of the sample preparation method. The coarse aggregate was deliberately avoided when the paste sample was obtained. This was to assure that the calcite of the aggregate was not mistaken for the calcite of carbonation. Also, by this method, the carbonation rims on the limestone were avoided. The rims on the tiny hydrous iron oxide pellets are not large and of only occasional occurrence. In any case, they would be harder than the paste and might have been rejected as aggregate when the paste sample was prepared. Minute pinpoints of calcite were easily observed with crossed polarized light and the scattered general distribution of this carbonation was obviously below the level of detection by X-ray diffraction.

CONDITION OF PASTE

PC# Location	Crossed by Continuous Crack	Type	General	High Water Zones	Micro-cracking	Bond Separation	Remarks
0597-S-1-B	No		very good	NS	NS	NS	
0599-S-3-A	Narrow	plastic	medium	common	minor	minor	
0602-S-4-A	Two major	plastic	very good	NS	NS	NS	plastic cracks cross
0603-S-4-B	Narrow	brittle	very good	NS	NS	NS	
0605-S-5-A	No		medium	minor	minor	minor	
0615-S-8-A	No		very good	NS	trace	trace	
0620-S-9-D	No		good	NS	trace	minor	
0621-S-10-A	No		medium	minor	minor	common	
0625-S-11-B	No		medium	minor	minor	common	
0629-S-12-D	No		poor	minor	common	common	cracks are up to two inches long
0630-S-13-Af	No		good	trace	minor	minor	
0631-S-13-Ap	No		very good	NS	NS	trace	high air content looks entrained
0638-N-1-C	No		good	NS	trace	minor	
0640-N-2-C	No		good	minor	trace	NS	
0641-N-3-A	No		poor	common	common	minor	
0642-N-5-A	No		very good	trace	NS	NS	
0645-N-6-C	No		good	minor	trace	NS	
0647-N-8-C	No		medium	minor	minor	minor	
0648-N-9-A	No		good	minor	minor	trace	the microcrackin may be extension of cracking abov
0649-N-9-C	No.		very good	NS	NS	trace	

CONDITION OF PASTE, continued ...

PC# Location	Crossed by Continuous Crack	Type	General	High Water Zones	Micro-cracking	Bond Separation	Remarks
0650-N-10-A	No		poor	very common	minor	minor	
0652-N-13-A	No		medium poor	minor	minor	common	minor wood trash
0653-N-13-C	No		poor	minor	common	common	
0654-N-14-A	No		poor	common	common	common	
0655-N-14-C	major	brittle	medium	minor	minor	common	may show evi-dence of plasti cracking at surface
0662-N-17-C	major	wood trash	very good	trace	NS	NS	major amount of vegetative trash
0663-6-6	No		good	trace	minor	minor	
0665-10-22	major	brittle	good	trace	trace	minor	may show evidenc of plastic crack at surface

NS = Not Significant