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16. Abstract

The purpose of this investigation is to provide administrators and designers with factual data on which to base decisions on the type of protection to provide to bridge decks constructed in corrosive environments. Specifically, the objective is to determine the relative time-to-corrosion of reinforcing steel embedded in concrete slabs, fabricated from various mix designs and construction procedures when the slabs are subject to periodic setting with a 3 percent NaCl solution.

One hundred and twenty-four, 4 ft. by 5 ft. by 6 in. reinforced concrete slabs were fabricated, cured, and placed in the outdoor exposure yard on elevated stands. Concrete covering the feasible range of mix designs were investigated. Clear concrete cover over the reinforcing steel was varied from 1 to 3 inches. Various finishing methods, curing procedures, admixtures, surface protective treatments and special new concretes were also included.

Volumes I and II, FHWA-RD-73-32 and 33, issued in April 1973 presented the details of slab fabrication and testing as well as an interim evaluation after 330 daily salt applications. This report, Volume 3, presents the results of evaluations of conventional concretes and those materials and techniques which "looked good" in the initial report. 830 daily saltings were applied prior to this evaluation.

17. Key Words

concrete, corrosion, half-cell, deicer, chloride penetration, mix design, corrosion testing, reinforcing steel, bridge deck

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PREFACE

One of the most severe problems facing the highway industry is chloride deicer induced reinforcing steel corrosion and the subsequent deterioration of concrete bridge decks. The Federal Highway Administration, Offices of Research and Development, consider elimination of bridge deck deterioration as one of their highest priority efforts. The problem and the approach to the solution of the problem are defined in Task 4B1 of the Federally Coordinated program of Transportation Research (FCP) under the title, "Elimination of Spalling and Delamination of Structural Concrete." The study described herein is one work unit within that task.

The author wishes to thank Mr. Richard E. Hay, Chief, Materials Division and Mr. Thomas J. Pasko, Chief, Paving and Structural Materials Group of the Federal Highway Administration for their aid in study design and reporting. Additionally, research technicians Appleton, Dwyer, Jones, Williams, Jackson, Nelson and Bailey deserve special thanks for their aid in slab fabrication, testing and data analysis.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

INTRODUCTION

One of the most severe problems facing the highway industry is chloride deicer induced reinforcing steel corrosion and the subsequent deterioration of concrete bridge decks. This bridge deck problem is shown in figure 1. The Federal Highway Administration, Offices of Research and Development, consider elimination of bridge deck deterioration as one of their highest priority efforts. The problem and the approach to the solution of the problem are defined in Task 4B1 of the Federally Coordinated Program of Research and Development in Highway Transportation (FCP) under the title "Elimination of Spalling and Delamination of Structural Concrete." One work unit within that task is the FHWA staff research study "Time-to-Corrosion of Reinforcing Steel in Concrete Slabs versus Various Parameters of Design and Construction."

The purpose of this investigation is to provide administrators and designers with factual data on which to base decisions as to the type of protection to provide for bridge decks constructed in corrosive environments. The specific study objective is: to determine the relative time-to-corrosion of reinforcing steel embedded in concrete slabs, fabricated from various mix designs and construction procedures, when the slabs are subjected to periodic wetting with a 3 percent sodium chloride solution. Interim findings, covering research performed on 120 4 ft by 5 ft by 6 inch (1.22 m by 1.52 m by 0.152 m) and four smaller reinforced concrete slabs, formed the basis of a two-volume interim report coauthored by K. C. Clear and R. E. Hay (1,2). These reports will be referenced as the initial study report throughout this report.

The initial report (1) provided in-depth information on the study research design, on the fabrication and testing of the slabs, and provided an interim evaluation of the many variables tested after approximately 330 daily applications of a 3 percent NaCl solution.

This report presents follow-up documentation on the performance of the original concrete slabs after a total of 830 daily salt applications as well as the performance of additional slabs after various numbers of salt applications. It deals predominately with conventional concretes and those materials which "looked good" in the initial report. Although a brief summary of slab fabrication and testing is presented, the reader should refer to the initial report for details.

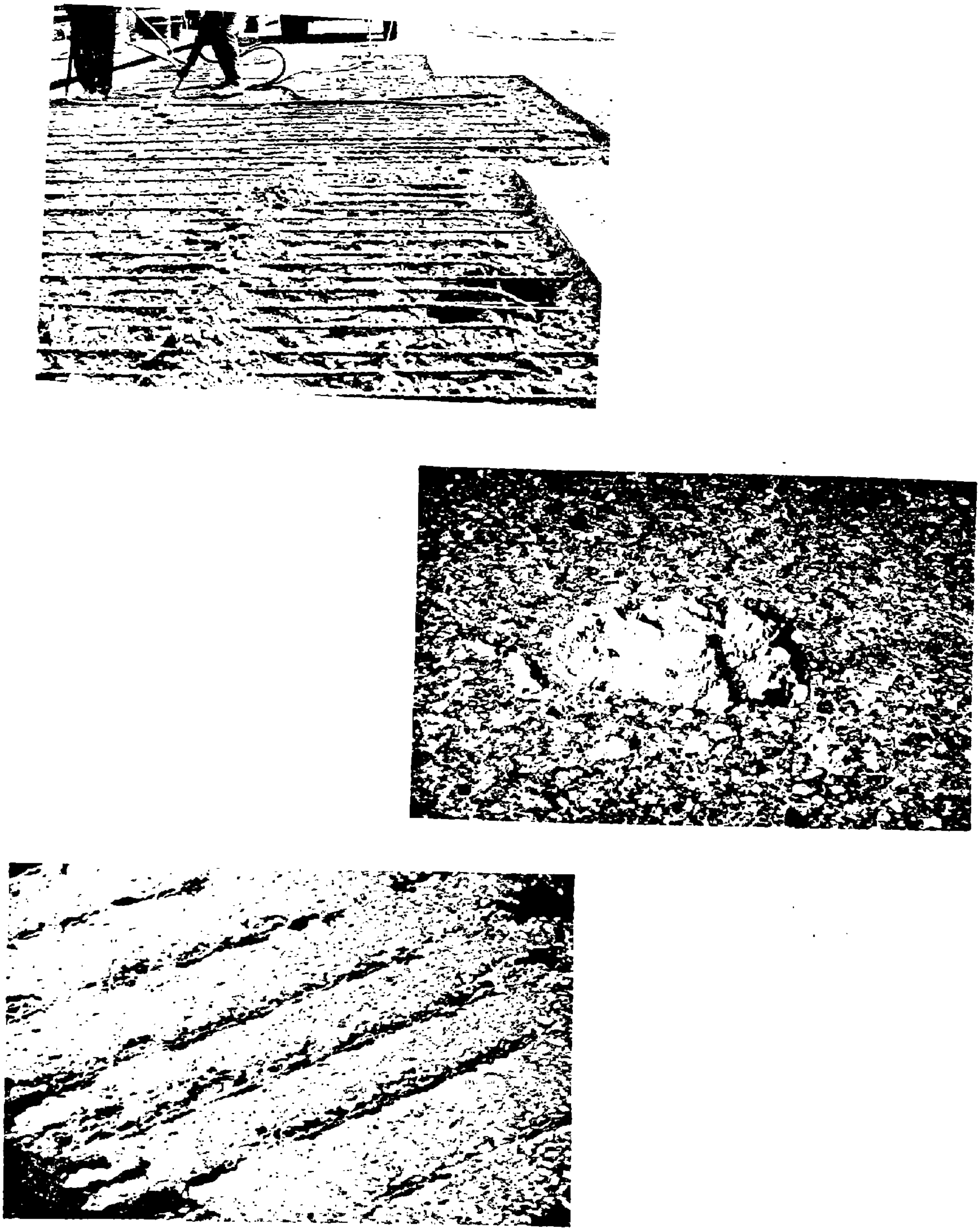


Figure 1. Bridge deck deterioration.

SUMMARY AND CONCLUSIONS

The purpose of this study is to provide administrators and designers with factual information on which to base decisions concerning the type of protection to provide bridge decks constructed in corrosive environments.

One hundred and twenty 20 ft^2($1.86m^2$) reinforced concrete slabs as well as several smaller slabs fabricated using various mix designs, construction techniques, and special treatments were subjected to daily ponding with a 3 percent sodium chloride solution. The performances of the slabs were determined using visual and delamination surveys, electrical half-cell potential monitoring, and coring and subsequent chloride analyses. The primary conclusions of this study are summarized below:

(1) The following have no apparent benefit as procedures for limiting chloride intrusion and subsequent rebar corrosion in uncracked concrete: Compacting with a steel trowel, surface revibration, chromate inhibitor, silicone admixture, expanded pumice as an admixture, varying the curing procedure (e.g., membrane, polyethylene sheet or wet burlap), an epoxy modified concrete, a periodic surface wash with a strong base (high pH) solution, a stearate surface treatment, increased cement factor without a decrease in water cement ratio, a ferro-cement mortar, the use of permanent steel bridge deck forms, and aging the concrete 1 year prior to salting.

(2) The following parameters will define the resistance of a conventional concrete to chloride ingress and subsequent rebar corrosion when acceptable construction practices are used.

(a) concrete water-cement ratio

(b) cover over the reinforcing steel

(c) degree of consolidation

The data show that a reduction in water-cement ratio from 0.50 to 0.40 by weight will yield about the same additional time-to-corrosion as a 1.0 inch (25 mm) increase in depth of cover. Further, experiences in slab fabrication indicate that good construction practices will provide an adequate degree of consolidation when the slump of the concrete is 3 inches (76 mm) or more. However, for lower slump concretes, a standard amount of vibration will not always insure proper consolidation and therefore, a consolidation monitoring device is recommended. Inadequate consolidation can completely negate the beneficial effects of an increase in cover and a decrease in water-cement ratio.

For the properly consolidated concretes, the minimum cover depths required to protect 95 percent of the reinforcing steel from chloride-induced corrosion through 830 salt applications in this test were:

w/c = 0.40 concrete - 1.7 inches (43 mm)

w/c = 0.50 concrete - 2.8 inches (71 mm)

w/c = 0.60 concrete - 3.1 inches (79 mm)

To translate these values to plan covers (i.e. the cover specified on bridge deck plans), one must take into account (1) the relation between plan cover and the average depth of cover achieved on bridge decks, (2) the variation in depth of cover which occurs during deck construction, (3) the permeability factor which should be used to translate the laboratory findings to the field, and (4) the field equivalency of 830 daily laboratory salt applications. Each of these factors is discussed in the body of this report.

Testing is in progress to define the field equivalency of time-to-corrosion saltings. After 1 year, the tests indicate that one time-to-corrosion daily salting is equivalent to somewhat less than one field salting. However, additional tests are necessary before this difference can be quantified.

(3) Polymer impregnation and the use of latex modified concrete are procedures which resulted in greatly decreased chloride intrusion. Use of the properly consolidated, low slump, low-water cement ratio (w/c=0.32) portland cement concrete commonly referred to as the "Iowa" mix also resulted in greatly decreased chloride intrusion. The minimum concrete cover the reinforcing steel required for protection against rebar corrosion through the indicated number of daily salt applications were:

Total polymer impregnation - 0.7 inches (18 mm)- 938 saltings

Surface polymer impregnation - 0.6 inches (15 mm)- 204 saltings

Latex modified concrete - 1.0 inches (25 mm)- 830 saltings

"Iowa" low slump concrete, w/c = 0.32, properly consolidated - 1.4 inches (36 mm)- 830 saltings

It should be noted that the covers given above are minimums and not plan covers. The factors listed in item (2) which affect translation of minimum rebar cover to plan cover should also be applied to these materials.

(4) The use of absorptive form liner or the use of vacuum treatment with the typical bridge deck concrete (w/c=0.5) will improve the concrete's resistance to chloride penetration to that of a 0.40 water-cement ratio concrete. However, no significant benefit with respect to chloride ingress resulted from the use of these techniques on a concrete with an initial water-cement ratio of 0.40.

FABRICATION AND TESTING

The test slabs in the outdoor exposure yard are shown in figure 2. All slabs are 4 ft by 5 ft by 6 inches (1.22 m by 1.52 m by 0.152 m) except for the polymer-impregnated concrete slabs which are slightly smaller.

The 20 sq ft (1.86 m^2) slabs were molded in watertight molds having a grid of No. 4 (13 mm diameter) reinforcing bars positioned at the required depth. The concrete was vibrated to 99 ± 1 percent of the rodded unit weight using a direct transmission nuclear density apparatus to control consolidation. The nuclear apparatus was a commercially-available, moisture-density gauge of the type commonly used to monitor soil compaction. Screeding was accomplished using a reproducible manual finishing process. The appropriate curing procedure was applied immediately after loss of surface sheen and remained on the concrete for 7 days.

After the slabs were removed from the molds, the sides were coated with epoxy resin to prevent water loss during testing. A small dike was placed around the outside edge of the top surface of each slab to permit ponding of the sodium chloride solution. Following 7 days of on-ground curing, the slabs were placed in an elevated position and leveled in an outdoor exposure yard. The surfaces were wire-brushed to remove the membrane curing compound before the initial sodium chloride application (applied at 6 weeks of age).

The top surface of each slab is subjected to ponding to a depth of 1/16 inch (1.6 mm) with a 3 percent sodium chloride solution each afternoon. Three evaluation techniques are used to determine the ability of each variable in preventing chloride migration to the level of the reinforcing steel and subsequent rebar corrosion:

(1) The electrical half-cell potential of the reinforcing steel is monitored with time by measurement of the rebar potentials at six predetermined positions on the surface of each slab and by determining the highest potential on each slab. The frequency of potential monitoring was once per week during the first year of salting and once every 2 weeks after the first year. Analysis yielded the following meaning of the potentials (referenced to the copper-copper sulfate half-cell, CSE).

   (a) Potentials consistently greater than 0.35 volts CSE - High probability of rebar corrosion.

   (b) Potentials consistently less than 0.20 volts CSE - High probability of no rebar corrosion.

   (c) Potentials in the range of 0.20 to 0.35 volts CSE - Uncertain area with regard to the condition of the reinforcing steel; may be active or passive. Use of another detection technique is indicated.

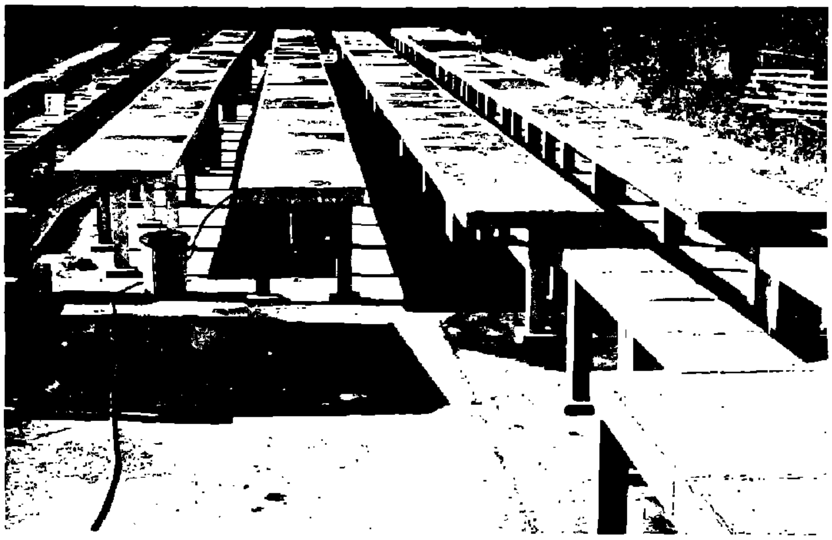


Figure 2. Time-to-corrosion slabs.

(2) The chloride content of the concrete is determined at select times by dry coring and analysis following the procedure described by H. A. Berman (3). Work by Lewis and studies in the FHWA laboratory have shown the chloride content corrosion threshold (i.e., the minimum quantity of chloride required to initiate rebar corrosion in a bridge deck portland cement concrete when sufficient moisture, oxygen, and other necessary factors are present) to be approximately 0.20 percent Cl^- per gram of cement (1,4). Thus, the total chloride content corrosion threshold for a specific concrete is dependent on the quantity of cement in that concrete. Although for field evaluations it may not be practical to have a threshold which is dependent on cement content, for research work where the cement content of each concrete is accurately known, such a procedure is feasible. Therefore, a variable threshold, based on the cement content of each concrete was used in this work. The total chloride thresholds for the various time-to-corrosion concretes are given in table 1.

Table 1. Total chloride corrosion thresholds.

Cement factor, 94 lb bags/yd^3	Cement content,[a] lbs/yd^3	Total Cl^- corrosion threshold,[b] lbs Cl^-/yd^3
6.0	564	1.13
6.75	634.5	1.27
7.0	658	1.32
8.0	752	1.50
8.75	822.5	1.65

[a] To convert lbs/yd^3 to kg/m^3 multiply by 0.594.

[b] To convert lbs Cl^-/yd^3 to parts per million (ppm) chloride by weight multiply by 255.4 (this conversion assumes a concrete unit weight of 3915 lbs/yd^3= 2325 kg/m^3 and is used throughout this report).

All chloride contents measured using the wet chemical analysis technique were adjusted to be representative of the concrete mass using the weight loss at 500 C procedure described in reference 4.

The baseline chloride content for "unsalted" time-to-corrosion concrete (i.e., the quantity of chloride in the materials) was 0.20 to 0.40 lbs Cl^-/yd^3 (102 ppm).

Note: The above figures differ from the baseline of 0.04 to 0.05 lbs Cl^-/yd^3 (10 to 13 ppm) reported in the initial report (1) because refined chloride analysis procedures are now being used for low chloride content concretes. The refinement, which is described on page 44 of reference 4, has resulted in better accuracy for low chloride concretes. However, the refinement has no effect on the results on samples with chlorides above baseline. Thus, the only chloride contents presented in the initial report which were erroneous were the baseline values.

(3) Visual and delamination surveys are frequently performed to detect the presence of rust stains, cracks, spalls, and hollow planes within the concrete.

Thus, a large quantity of data is available for determination of the condition of the test slabs with respect to corrosion of the reinforcing steel. In addition to the information from the visual and delamination surveys, over 75,000 half-cell potential measurements have been obtained and more than 1200 chloride analyses have been performed to document the chloride versus depth profiles within the slabs.

FINDINGS OF THE INVESTIGATION

In this follow-up report, the major purposes are to verify, modify or reject the conclusions of the initial report and to provide longer-term test information on those mix designs, construction techniques, and special treatments which showed promise in the initial evaluation. Briefly, the major conclusions of the initial report were:

(1) The typical bridge deck concrete (w/c=0.50, CF=7) provides very little protection against chloride induced corrosion when the cover over the reinforcing steel is only 1 inch (25 mm).

(2) The water-cement ratio of a concrete appears to be the primary determinant of its ability to resist chloride penetration. Conversely, cement factors in the range of 6 to 8 bags per cubic yard (564 to 752 lbs/yd^3 -335 to 446 kg/m^3) have little effect when studied at a constant water cement ratio.

(3) Adequate consolidation to at least 98 percent of the rodded unit weight of the concrete is a prerequisite to the construction of a chloride resistant bridge deck.

(4) Minimum depths of covers over the reinforcing steel of at least 2 inches (51 mm) for a 0.40 water-cement ratio concrete and 3 inches (76 mm) for a 0.50 water cement ratio concrete will undoubtedly be required if long-term protection is desired from conventional concrete.

(5) Polymer impregnated concrete, latex modified concrete, and the use of absorptive form liners or vacuum treatment provide a significant increase in resistance to chloride penetration.

(6) The following techniques are of no apparent benefit as means of reducing chloride intrusion and rebar corrosion in uncracked concretes: compacting with a steel trowel, surface revibration, chromate inhibitor, silicone admixture, expanded pumice as very fine aggregate, ferro-cement, curing procedure-membrane, wet burlap or polyethylene sheet, an epoxy modified concrete, and a periodic surface wash with a strong base.

As stated above, techniques which provide little or no additional protection will not be discussed in depth herein. Rust stains on the slab surfaces and high electrical potentials have continued to provide evidence that the above list of techniques which have no apparent benefit is correct.

All chloride analyses performed after 830 salt applications were done at various depths within each slab to yield a chloride versus depth profile for each concrete. In the interest of brevity, the results of each of the 1000 analyses are not listed herein but rather plots of average and maximum chlorides versus depth are presented in the body of this report and a table summarizing the data is presented in the Appendix.

Throughout the discussion sections of this report, threshold depths will be discussed. The threshold depth is defined as the depth within a particular concrete that the chloride profile (average, maximum, or 95 percent level) crossed the chloride threshold when straight line interpolation between the data for successive sampling depths was used. The sampling depths used were 1/16 to 1/2 inch (2 to 13 mm), 3/4 to 1-1/4 inches (19 to 32 mm), 1-3/4 to 2-1/4 inches (44 to 57 mm), 2-3/4 to 3-1/4 inches (70 to 83 mm) and 3-3/4 to 4-1/4 inches (95 to 108 mm) in all instances, and the mean depths of 0.28, 1.0, 2.0, 3.0, and 4.0 inches (7, 25, 51, 76 and 102 mm) were used as the plotting points for profile plots. Thus, if, for slab X, the maximum chloride content at mean depth 2.0 inches (51 mm) was 5.0 lbs/yd^3, the maximum Cl^- at depth 3.0 inches (76 mm) was 0.5 lbs/yd^3, and for this slab the Cl^- threshold was 1.3 lbs/yd^3 (332 ppm), the maximum chloride threshold depth would be:

$$2.0 \text{ inches} + \frac{5.0 - 1.3}{5.0 - 0.5}(1.0 \text{ inches}) = 2.0 + 0.8 = 2.8 \text{ inches } (71 \text{ mm})$$

If all the steel in that slab had been placed at depths greater than 2.8 inches (71 mm) no chloride induced corrosion would have occurred. The straight line interpolation procedure was used rather than obtaining the threshold depths from plots because it results in a slightly conservative value.

Effect of Water Cement Ratio and Depth of Cover

To define the effect of water-cement ratio and depth of cover over the reinforcing steel, slabs were fabricated using concretes with water cement ratios (w/c) of 0.40, 0.50, and 0.60 by weight, a constant cement factor of 7.0 (658 lbs/yd^3-390 kg/m^3) and cover depths of 1, 2, and 3 inches (25, to 51 and 76 mm). Only nominal reinforcement, No. 4 bars on 12 inch (0.3 m) centers, was used in these slabs.

The poor performances of the typical bridge deck concrete (w/c=0.50) with 1 inch (25 mm) cover and the 0.60 water-cement ratio concrete with 1 inch (25 mm) cover were documented in the initial report (1). After 330 salt applications, rust stains were visible although no cracking had occurred. Electrical potentials and chloride analyses indicated the initiation of corrosion in 7 to 28 days.

Through the additional 500 daily saltings since the initial report, visible rust stains have become more numerous and short cracks have occurred on several of the 0.60 water-cement ratio slabs with 1.0 inches (25 mm) cover. Electrical potentials have continued to be in the active corrosion (>0.35 volts CSE) or the inconclusive (0.20 to 0.35 volts CSE) ranges for all 0.50 and 0.60 water-cement ratio slabs with 1.0 inches (25 mm) of cover. Most potentials are in the inconclusive range for the 0.50 and 0.60 water-cement ratio slabs with 2 and 3 inches (51 and 76 mm) of cover. Similarly, the potentials are inconclusive for the 0.40 water-cement ratio concrete of all cover depths. Rust stains are now visible on several 0.40 w/c slabs with 1.0 inches (25 mm) of cover but no cracking has occurred. Similarly, no cracking has yet occurred on the lightly reinforced slabs with 0.50 water-cement ratio although numerous rust stains are present. Figures 3 and 4 show the average and maximum chloride profiles obtained by analyzing 262 samples from the 3 water-cement ratio concretes. The data confirm that water-cement ratio is a major factor in determining the resistance of a conventional concrete to chloride intrusion when the cement content is in the range of 6 to 8 bags/yd^3.

The data can best be summarized by calculating a cover depth required to protect the reinforcing steel from intrusion of sufficient chloride to induce corrosion. These depths (Tmax), based on the maximum chloride encountered at each test depth are presented in table 2 for both 330 and 830 salt applications. For the 830 salt application data, a T_{95} depth defined as the depth of cover required to protect 95 percent of the reinforcing steel from chloride-induced corrosion is also presented. The procedure used to calculate the T_{95} value is discussed in the Appendix.

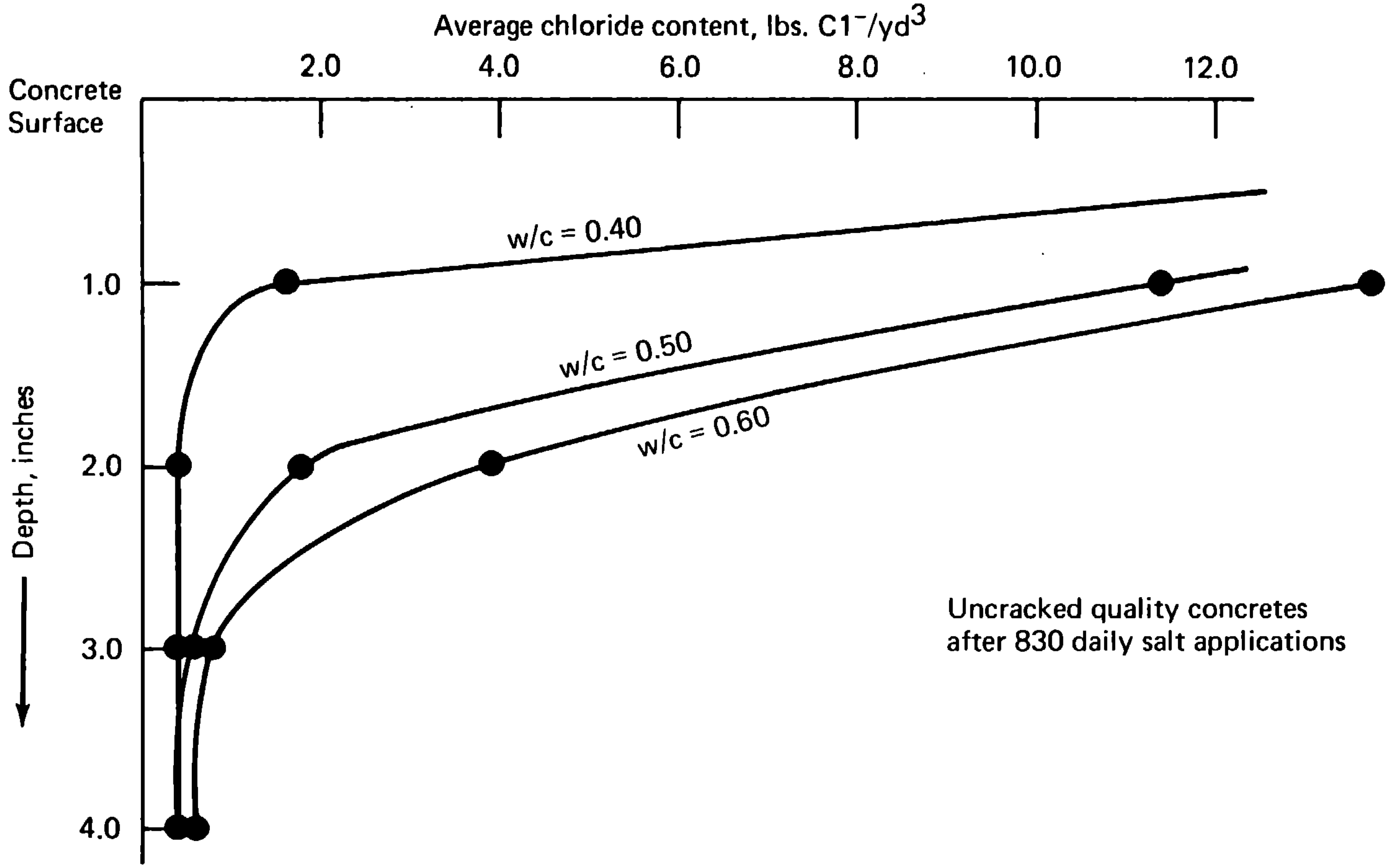


Figure 3. Effect of water cement ratio — Average chloride profiles.

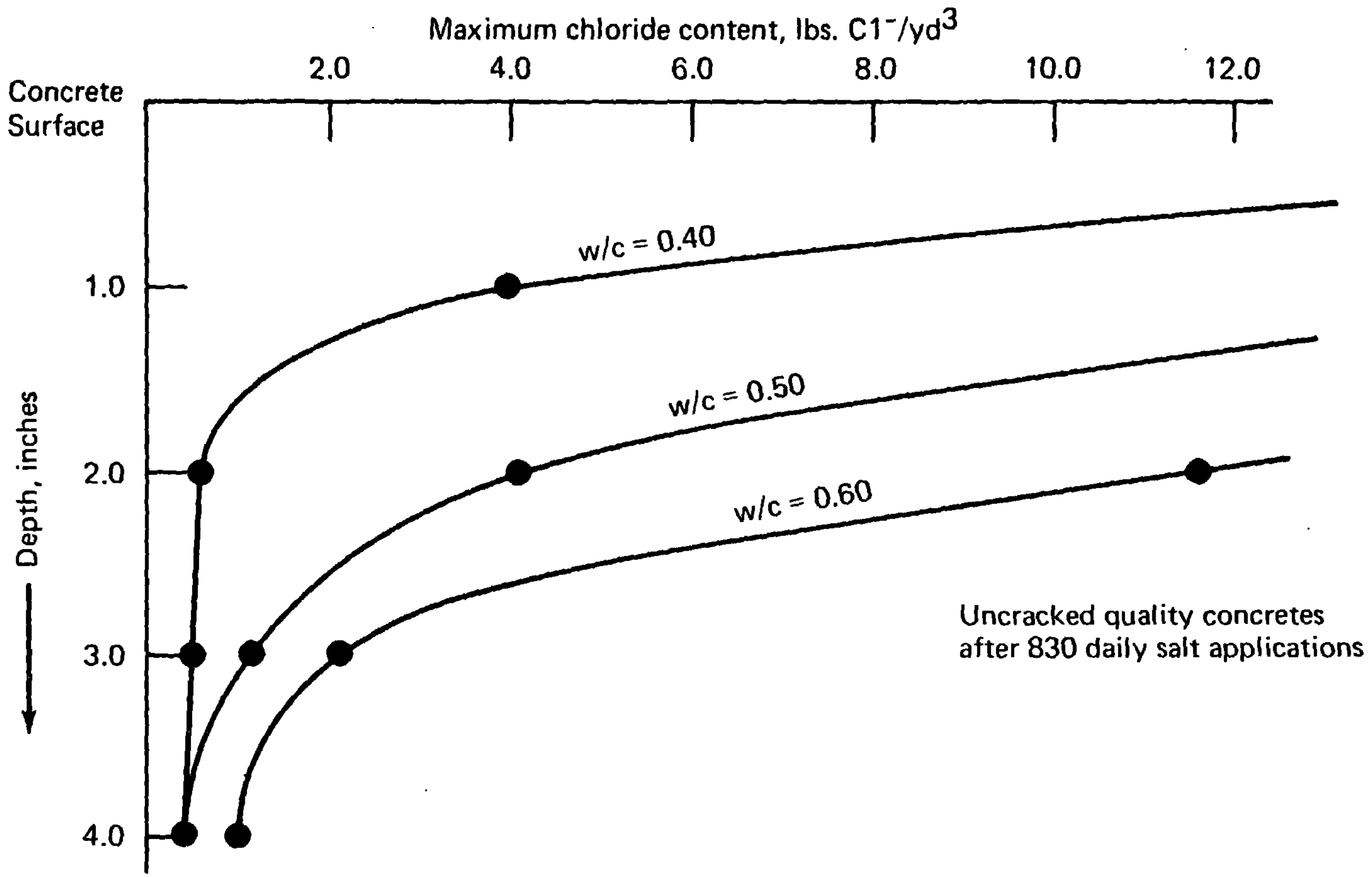


Figure 4. Effect of water cement ratio – Maximum chloride profiles.

Table 2. Depth of concrete cover required.

Concrete	Tmax after 330 salt applications, inches[a]	Tmax after 830 salt applications, inches[a]	T_{95} after 830 salt applications, inches[a]
w/c=0.4	1.3	1.8	1.7
w/c=0.5	2.5	2.9	2.8
w/c=0.6	2.9	3.7	3.1

[a] To convert inches to mm, multiply by 25.4.

Data in the table show that about twice as much cover is required to protect reinforcing steel in a 0.60 w/c concrete than is required for a 0.40 w/c concrete. Also, a 0.50 w/c concrete will require about 1 inch (25 mm) more cover over the steel than a 0.40 w/c concrete.

The tremendous effect that cover depth has on the time-to-corrosion in a concrete of any of the water-cement ratios is demonstrated in figure 5. The time-to-corrosions used to obtain the plots of cover depth required versus number of daily saltings were obtained by using all the available data (rust stains, electrical potentials, and chloride analyses). The estimated time-to-corrosions for each concrete for 1.0, 2.0, and 3.0 inch (25, to 51 and 76 mm) depths of cover (obtained from figure 5) are shown in table 3. Data in the figure and the table are estimates in that it is impossible to determine such things as how much time elapses between the initiation of corrosion and the appearance of a rust stain when electrical potentials are inconclusive. However, they are undoubtedly correct with respect to order of magnitude and serve the purpose of illustrating the relative time-to-corrosion.

Table 3. Effect of water-cement ratio and depth of cover on time-to-corrosion.

Depth of cover, inches[a]	Estimated Time-to-Corrosion, days of salting w/c=0.40 concrete	 w/c=0.50 concrete	 w/c=0.60 concrete
1.0	120	7 to 28	7
2.0	1000	140	75
3.0	>> 1000	925	375

[a] To convert inches to mm, multiply by 25.4.

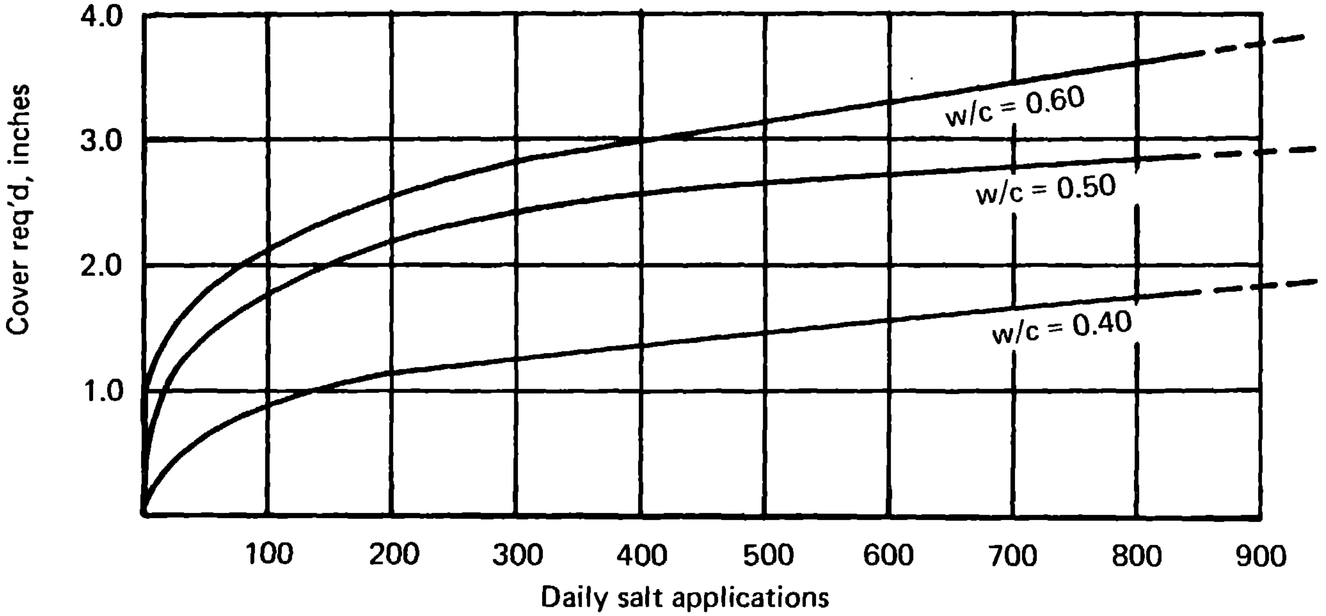


Figure 5. Effect of water cement ratio and depth of cover on relative time to corrosion.

Effect of Cement Content

To isolate the effect of cement content on the chloride permeability of concrete, three mix designs all with water-cement ratios of 0.50 but with different cement contents (6, 7, and 8 bags/yd^3 - 564, 658 and 752 lbs/yd^3-335, 390 and 446 kg/m^3) were used. All slabs fabricated using these mix designs had 1.0 inch (25 mm) cover over the reinforcing steel.

High electrical potentials and rust stains indicate that corrosion is occurring in the slabs with all three cement factors.

Figures 6 and 7, based on 146 chloride analyses, show that the average and maximum chloride profiles for the concretes with all three cement factors are all virtually identical regardless of cement content except at the 1 inch (25 mm) depth. At this depth, somewhat lower chlorides were found in the lowest cement factor concrete than were found in those concretes with higher cement factors. Perhaps this was caused by the lower shrinkage associated with this concrete or perhaps it was merely test variability. However, the difference virtually disappears when one calculates maximum threhold depths since the lower cement factor concrete requires less chloride to induce corrosion. The threshold depths for the three concretes with 0.50 w/c ratios are:

CF = 6.0 - 2.7 inch (69 mm) <u>maximum</u> threshold depth

CF = 7.0 - 2.9 inch (74 mm) <u>maximum</u> threshold depth

CF = 8.0 - 2.8 inch (71 mm) <u>maximum</u> threshold depth

The maximum chloride profile and the maximum threshold depth for the concrete with a cement factor of 7 (658 lbs of cement/yd^3-390 kg/m^3) are slightly below those for the other profiles. This is probably due to the fact that three times as many samples were taken from that concrete. Consequently, the maximum chlorides found were more closely equal to the maximum throughout the population than for the other concretes which were sampled less.

Thus, data for 830 salt applications indicate that although a slight difference may exist between the concretes of different cement factors, no significant reduction in chloride induced corrosion can be achieved by only changing the cement content.

Effect of Inadequate Consolidation

Experiences during fabrication of the time-to-corrosion slabs indicated that good consolidation could be achieved simply through the use of internal vibration and the judgement of experienced personnel when the slump of the concrete was 3 inches (76 mm) or more. However, with low-slump concretes, good construction practices could not be relied upon to yield adequate consolidation. Data presented in the initial report

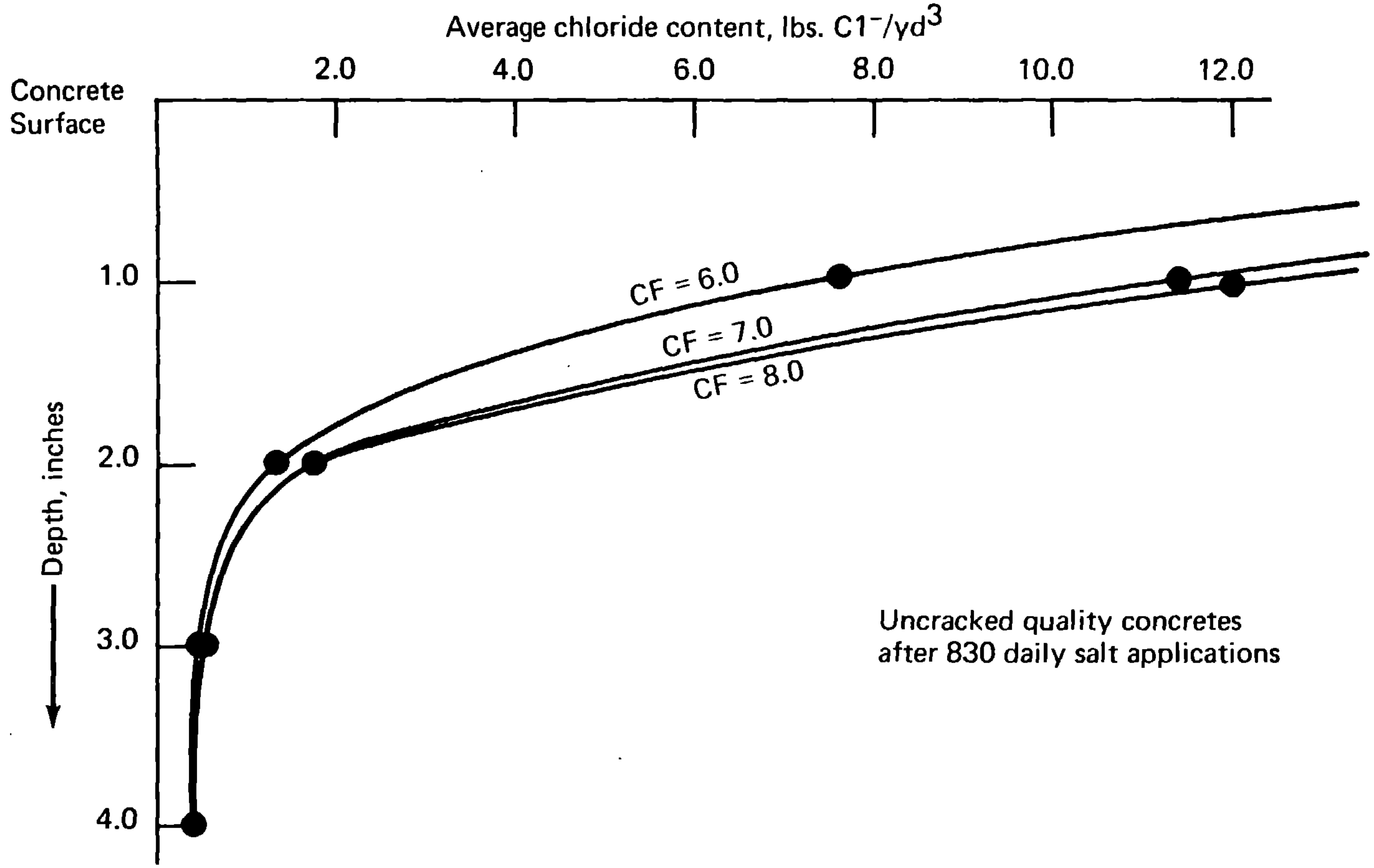


Figure 6. Effect of cement content — Average chloride profiles.

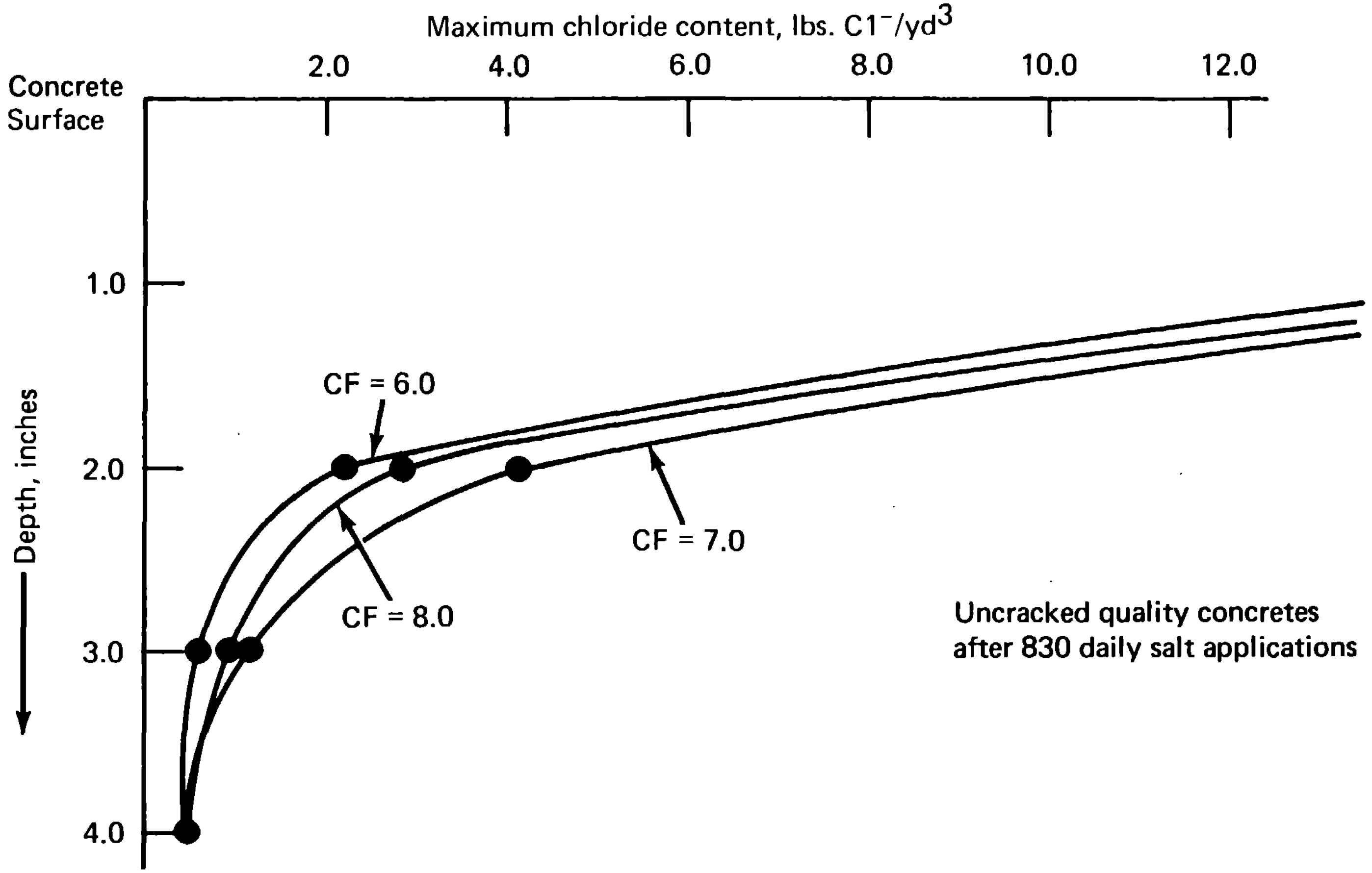


Figure 7. Effect of cement content – Maximum chloride profiles.

showed that a greater than nine-fold decrease in relative time-to-corrosion of reinforcing steel placed at a 1 inch (25 mm) depth resulted due to inadequate consolidation of a low slump, low water-cement ratio concrete.

Figure 8, based on 60 chloride analyses, shows the average and maximum chloride profiles within the properly consolidated and the inadequately consolidated low water cement ratio "Iowa" mix concretes (w/c=0.32, CF=8.75=822.5 lbs/yd^3= 488 kg/m^3, slump=0.5 inch=13mm) after 830 daily saltings. Obviously, there is a drastic difference in the two concretes of like mix design. The chloride profile for properly consolidated concrete is based on chloride analyses of samples from various depths within two slabs with in-place densities of about 98 percent of the concrete's rodded unit weight whereas the profile for improperly consolidated concrete represents the chlorides in two slabs with in-place densities of 92 to 94 percent of the rodded unit weight. The in-place densities were determined on two slabs by using a direct transmission nuclear density gauge during slab fabrication and on all four slabs by determining the unit weight of 4 inch (102 mm) diameter cores.

The maximum threshold depths for the two concretes which differ only with respect to degree of consolidation were:

"Iowa" low slump, low water-cement ratio concrete -
properly consolidated - 1.4 inches (36 mm)

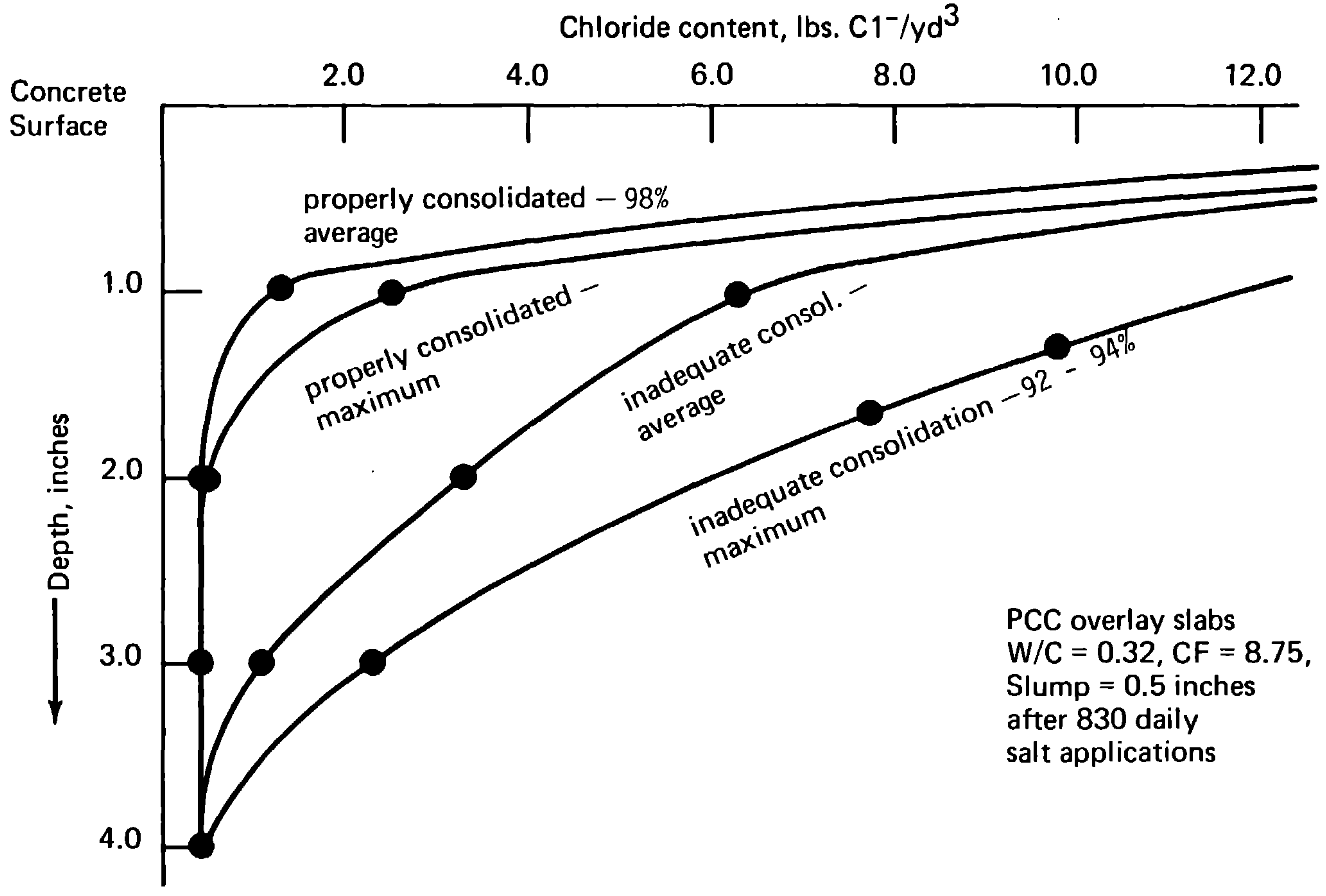
"Iowa" low slump, low water-cement ratio concrete -
improperly consolidated - 3.4 inches (86 mm)

Thus, the cover over the reinforcing steel required for protection against rebar corrosion through 830 salt applications was increased by 2 inches (51 mm) simply due to the 4 to 6 percent (about 6 to 9 lbs/yd^3- 3.6 to 5.3 kg/m^3) reduction in in-place density. The 3.4-inch (86 mm) requirement is 2.5 times that required for the same 0.32 water-cement ratio "Iowa" concrete when the concrete is properly consolidated and is almost equal to the 3.7 inches (94 mm) required for a 0.60 water-cement ratio, high slump concrete.

The maximum chloride threshold depth of 1.4 inches (36 mm) for the 0.32 water-cement ratio, properly consolidated low slump "Iowa" concrete, is slightly less than the 1.8 inch (46 mm) depth needed in a properly consolidated 0.40 water cement ratio portland cement concrete. This lesser depth requirement would be expected in light of the effect of water-cement ratio discussed previously.

Effect of Concrete Age at First Salting

Generally, all the concrete slabs used in this research received their first salt application exactly 6 weeks after fabrication. However, a single 20 ft^2(1.86 m^2) slab fabricated using the w/c=0.50, CF = 7



PCC overlay slabs
W/C = 0.32, CF = 8.75,
Slump = 0.5 inches
after 830 daily
salt applications

Figure 8. Effect of inadequate consolidation — PCC overlay slabs.

concrete mix design and 1 inch (25 mm) of concrete cover over the reinforcing steel was "aged" in an outdoor environment 1 year prior to salting.

Although the half cell potentials on this slab have generally remained in the inconclusive range, rust stains which were first seen on the slab surface 10 months after the initiation of salting, confirmed that corrosion was occurring. The chloride profile within the slab was determined after 600 salt applications using 15 samples from various depths. The depth within the aged concrete that the maximum chloride profile crossed the corrosion threshold was 3.2 inches (81 mm), not significantly different from the 830 salt application maximum of 2.9 inches (74 mm) for the early salted 0.50 water-cement ratio slabs.

Thus, these data indicate that allowing the concrete to "age" 1 year prior to salting will not increase its resistance to chloride ingress.

Corrosion Causes Cracking and Spalling

To provide a time to detrimental corrosion (i.e., time to concrete cracking) under severe conditions for the typical bridge deck concrete, several slabs were fabricated with a reinforcing mat of No. 5 (16 mm diameter) bars on 2 inch (51 mm) centers rather than the normal pattern of No. 4 (13 mm diameter) bars on 12 inch (0.3 m) centers used in most slabs. A bottom rebar mat was also placed in the heavily reinforced slabs. The 3 inch (76 mm) slump, typical air-entrained bridge deck concrete (w/c=0.5, CF=7) mix was used with 1 and 2 inch (25 and 51 mm) rebar covers.

Electrical potentials indicated corrosion was quickly initiated (seven salt applications) in the slabs with 1 inch of rebar cover and the surfaces were spotted with rust stains 3 months after the initiation of salting. Widespread cracking, the full length of the 5 ft (1.5 m) slabs occurred about 15 months after the initiation of corrosion and small spalls and delaminated areas soon followed. Figures 9 and 10 show the heavily reinforced slabs with 1 inch (25 mm) cover and the cracking and delamination caused by corrosion. This finding, that corrosion can induce distress in less than a year and a half on slabs which have never been subjected to dynamic loading, should dispell any lingering beliefs that spalling is caused by live loads or the flexibility of bridges.

The similar slabs with 2 inches (51 mm) of cover have not cracked as yet (27 months after initiation of salting) although electrical potentials and rust stains indicate that the reinforcing steel is corroding.

Relation of Daily Salting to Field Time-to-Corrosion

Although this study is aimed at providing only relative information concerning the resistance of the various concretes and special treatments to chloride intrusion and rebar corrosion, the question "What is the field equivalency of X number of salt applications?" often arises. Such a question is extremely difficult to answer.

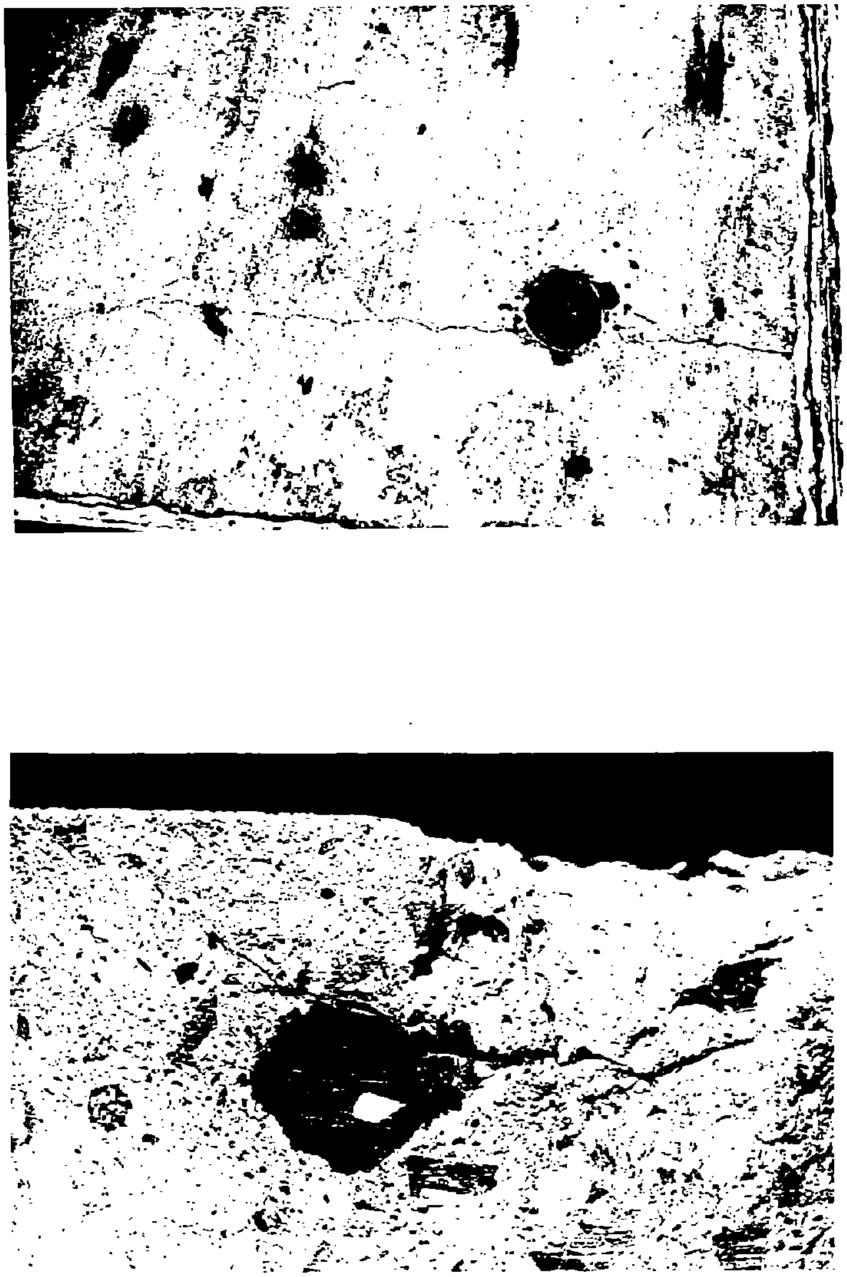


Figure 9. Corrosion caused cracking in the heavily reinforced slabs.

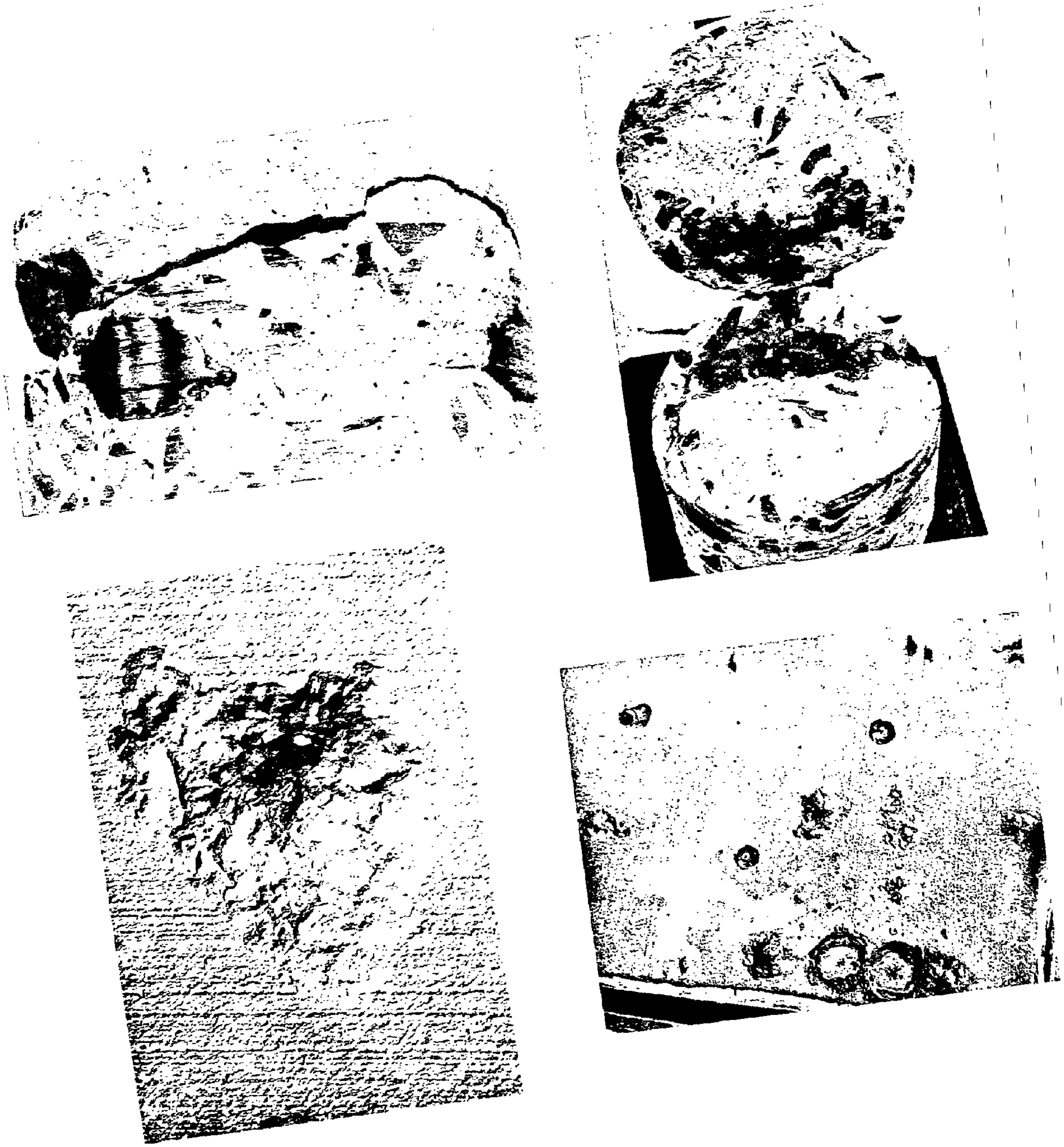


Figure 10. Corrosion caused delamination and spalling in the heavily reinforced slabs.

By salting the slabs daily, a certain acceleration factor is undoubtedly achieved. However, it probably is not complete acceleration such that one time-to-corrosion salting is equivalent to a single deicing salt application on any bridge deck. Our work has indicated that there are two mechanisms of chloride migration in concrete. One is the intrusion of chloride bearing water into the unsaturated concrete due to capillary action and gravity. This is undoubtedly accelerated significantly by the daily ponding procedure used in the time-to-corrosion work; perhaps even such that one daily salting is equivalent to one field salting for the Washington, D.C. environment since a wet/dry slab surface condition is achieved on most days.

The other mechanism for chloride movement through concrete is ion migration through the water already in the concrete. Ion migration occurs because the chloride ions desire to achieve chemical equilibrium (i.e., equal distribution throughout the concrete). This mechanism is dependent upon the moisture content of the concrete (i.e., without water no ion movement will occur) and time. In our environment, the daily ponding procedure probably does little to accelerate this mode of migration and thus for this mechanism, one daily salt application is probably more closely equal to 1 day in the life of a bridge (winter or summer) rather than one application of deicing salt.

The relative importance of the two mechanisms will be dependent on the moisture content of the concrete both when salt is applied and throughout its life. A dry concrete when salted will absorb water bearing chloride rapidly. If the concrete remains wet, ion migration will occur even without additional salting. However, if the concrete drys out, no ion migration will occur until it is rewetted.

Conversely, a concrete which is saturated when salted will gain chloride only by ion migration through the water already in the concrete. This ion migration will continue as long as moisture is present.

Because of the dependency of chloride ingress on moisture content and time and because of the many variable salting rates and saltings per year used on bridge decks it will be difficult to define a universally applicable formula relating a time-to-corrosion salting to a "field salting." However, perhaps the question can be answered "Are 830 time-to-corrosion salt applications more closely equal to 2, 10, 20, 40, or 100 years in the life of a lightly salted (or a heavily salted) bridge?"

Toward this end, 28- 2.0 ft x 2.5 ft x 6 inch (0.61 m x 0.76 m x 0.152 m) concrete slabs were fabricated during the summer of 1974 using the 0.40 and 0.50 concrete mix designs and the standard time-to-corrosion fabrication procedures (1). These slabs are being subjected to the following different rates of salting:

(1) Standard time-to-corrosion salting for various periods - ponding daily to a 1/16 inch (1.6 mm) depth with a 3 percent sodium chloride solution.

(2) Salting with rock salt (NaCl) only when ice or snow is on the slab surface. No dams to retain the melted snow or ice were placed on these slabs. The amount of "chloride" applied per salting was the same as that used in the ponding work and translated to 620 lbs of rock salt per lane mile (175 kg/lane km).

(3) Ponding to a 1/16 inch (1.6 mm) depth with the 3 percent NaCl solution twice per week during the period December 1 through February 28 (i.e., 26 saltings per year).

Initially, it was necessary to establish the correlation of the 2.0 x 2.5 ft (0.61 m x 0.76 m) slabs with the larger 20 ft^2 (1.86 m^2) time-to-corrosion slabs. The cement source had changed and the environmental conditions could not be expected to be identical to those seen in 1972. To accomplish this, chloride profiles within the 0.40 and 0.50 water-cement ratio small concrete slabs were determined after 330 daily applications of a 3 percent NaCl solution. The maximum chloride threshold depths were:

w/c 0.40 concrete (small slabs) - 1.3 inches (33 mm)

w/c 0.50 concrete (small slabs) - 2.7 inches (69 mm)

These are similar to the maximum chloride threshold depths of 1.3 and 2.5 inches (33 and 64 mm) found within the larger 0.40 and 0.50 w/c time-to-corrosion slabs after 330 daily saltings. Thus, the original time-to-corrosion data have been confirmed and data obtained from the smaller slabs can be translated directly to the larger time-to-corrosion slabs.

The testing of the periodically salted slabs will be continued for at least another year before conclusions concerning the field equivalency of time-to-corrosion saltings will be available. However, chloride profiles obtained after the first year's testing have indicated certain tendencies which are presented below. Although the slabs from both water-cement ratio concretes were studied, chloride ingress into the 0.40 water-cement ratio slabs was low in all instances. Therefore, only the 0.50 w/c slab information will be used in this interim evaluation. As in the other sections of this report, chloride threshold depths will be used as a basis for comparison of the different salting procedures and a summary of the chloride data is presented in the Appendix.

The slabs which received rock salt applications only when snow or ice was present on their surfaces were salted eight times during the winter. Chloride profiles were determined in October 1975, 8 months after the last salting, to define the effect of "1 year" of light field exposure. The maximum chloride threshold depth for the rock salted slabs with a water-cement ratio of 0.50 was 0.96 inches (24 mm).

As discussed previously, seven time-to-corrosion saltings were the minimum required to induce corrosion of the reinforcing steel placed at a 1 inch (25 mm) depth within the 0.50 water-cement ratio large slab concrete. Or said another way, after seven daily salt applications, the maximum chloride threshold depth for the 0.50 water-cement ratio concrete was 1.0 inches (25 mm). This finding was confirmed by determining the chloride profile within daily salted 2.0 x 2.5 ft (0.61 x 0.76 m) slabs with a 0.50 w/c ratio immediately after 8 applications of a 3 percent NaCl solution. The maximum threshold depth was 0.97 inches (25 mm), not significantly different from the 0.96 inches (24 mm) found in the rock salted concrete of the 1.0 inches (25 mm) after seven salt applications found in the original time-to-corrosion work.

Thus, the initial eight daily time-to-corrosion applications of a 3 percent NaCl solution are approximately equivalent to the initial eight rock salt applications to remove ice and snow. It appears that in this instance, ion migration was not an important consideration. However, perhaps this could be explained by the fact that the differential chloride between various depths was relatively small because the concrete received only eight salt applications.

The slabs which had been winter salted 26 times (twice per week from December through February) were also cored 8 months after the final salting. The maximum chloride corrosion threshold for the 0.50 water-cement ratio concrete was 1.5 inches (38 mm). Conversely, similar slabs which were cored immediately after 26 daily applications of a 3 percent NaCl solution exhibited a maximum chloride corrosion threshold depth of 1.3 inches (33 mm). The average chloride corrosion threshold for the winter salted slabs was 1.2 inches (30 mm) whereas the average threshold for slabs cored immediately after daily ponding was 0.98 inches (25 mm).

Thus, these data indicate that winter salting followed by 8 months of exposure in the atmosphere is somewhat more severe than an equivalent number of daily salt applications. A direct relation between the two maximum threshold depths would indicate that one time-to-corrosion salting is equivalent to 1.3/1.5 or 0.87 field saltings. Similarly, based on average threshold depths, one time-to-corrosion salting would equal 0.98/1.2 or 0.82 field saltings. In this instance, comparison of the average threshold depths is undoubtedly the better procedure since the average is a better indicator of the population than the maximum and the absolute value of the threshold is not important. However, a direct relation may not be valid since the plots of cover required versus number of daily salt applications presented in figure 3 are not linear. The 0.50 w/c curve in figure 3 indicates that about 11 additional saltings would be required to change the threshold depth from 0.98 to 1.2 inches (25 to 30 mm). In other words, 37 daily time-to-corrosion saltings would be equivalent to 26 "winter" saltings; a ratio of 1 to 0.70. The cause of the lesser effect of the daily saltings in this test is undoubtedly ion migration during the 8 months after the final salting. Unlike the slabs which were salted only eight times, large quantities of chloride were available for downward migration within the concrete which had been salted 26 times.

Thus, in summary, the interim data indicate that one time-to-corrosion salting is equivalent to somewhat less than one field salting. The differences in chloride threshold depths are small and require confirmation by additional tests before further conclusions can be made.

Conventional Concrete - Discussion

The time-to-corrosion data indicate that the following items will define the performance life of a conventional concrete with respect to rebar corrosion on a bridge deck when acceptable construction practices are used.

(1) concrete water-cement ratio

(2) cover over the reinforcing steel

(3) degree of consolidation

The data show that a reduction in water-cement ratio from 0.50 to 0.40 by weight will yield about the same additional time-to-corrosion as a 1.0 inch (25 mm) increase in depth of cover. Further, experiences in slab fabrication indicate that good construction practices will provide an adequate degree of consolidation when the slump of the concrete is 3 inches (76 mm) or more. However, for low slump concretes, a standard amount of vibration will not always insure good consolidation and, therefore, a consolidation monitoring device is recommended. Inadequate consolidation can completely negate the effect of an increase in cover and a decrease in water-cement ratio.

For the properly consolidated concretes, the minimum cover depths required to protect 95 percent of the reinforcing steel from chloride-induced corrosion through 830 salt applications in this test were:

w/c = 0.40 concrete - 1.7 inches (43 mm)

w/c = 0.50 concrete - 2.8 inches (71 mm)

w/c = 0.60 concrete - 3.1 inches (79 mm)

To translate these values to plan covers (i.e. the cover specified on bridge deck plans), one must take into account (1) the relation between plan cover and the average depth of cover achieved on bridge decks, (2) the variation in depth of cover which occurs during deck construction, (3) the permeability factor which should be used to translate the laboratory findings to the field, and (4) the field equivalency of 830 daily laboratory salt applications.

Several pachometer (a magnetic instrument for the nondestructive determination of cover depth) surveys have recently been made throughout the country. New York researchers (5) concluded that "a tolerance of ± 1/2 inch (13 mm) should be attainable 95 percent of the time where 'good' construction practices are observed." This research also showed that the mean depth of cover on 50 spans was only slightly greater than the the plan cover of 2 inches (51 mm). New Jersey research (5) found the mean cover depth to be slightly less than the plan cover for decks with 2 inch (51 mm) design cover. Conversely, the mean depth was slightly more than the plan cover on decks with a 1.5 inch (38 mm) design cover. The standard deviation of concrete cover in New Jersey's survey was 3/8 inch (9.5 mm) and thus, for 95 percent of the steel to be below a mean depth of 2.0 inches (51 mm), it would be necessary to specify a 2.65 inch (67 mm) plan cover depth.

A nationwide survey of concrete cover on bridge decks was recently completed by an FHWA demonstration projects team (7). The pertinent survey findings were:

(1) The distribution of cover depth was equally distributed around the average or only slightly skewed.

(2) Typically, the plan cover and the average cover were almost equal.

(3) To obtain a minimum cover 90 percent of the time, the plan cover must be 1/2 to 5/8 inches (13 to 16 mm) greater than the minimum desired.

Therefore, data from all three studies indicate that the mean cover depth is usually approximately equal to the plan cover and that because of variability, to protect 90 to 95 percent of the steel, the plan cover must be 0.5 to 0.7 inches (13 to 18 mm) greater than the minimum required depths given above for the three water-cement ratio concretes.

A third item to consider when translating the laboratory findings to bridge deck plan rebar covers is the permeability factor. The water-cement ratio of bridge deck concrete is undoubtedly more difficult to control than the w/c of laboratory concrete. Thus, the probability of exceeding a specified maximum water-cement ratio is greater in the field. Similarly, proper consolidation of all concrete is less certain in field construction. Because of the above variables, a permeability factor should be applied to the minimum cover depths determined in the laboratory. In other words, it is recommended that the minimum cover depth be increased by at least 20 percent (permeability factor = 1.2) to account for variability in field construction. It should be noted that the permeability factor has no relation to and does not overlap the normal factor of safety applied by the designer to concrete strength.

The final item listed above as affecting the "plan" cover required for long lasting bridge decks is the field equivalency of 830 daily salt applications. As discussed earlier, although tests are underway, there is no definitive data available to relate time-to-corrosion laboratory saltings to field bridge deck saltings. Interim data indicate that one laboratory salting is not equivalent to one field salting, but rather a laboratory salting is not as severe an exposure as one typical field salting. Since more definitive information is not available, the user must exercise judgment when relating the above minimum cover values to bridge decks which are expected to perform without corrosion caused maintenance for 40 or more years.

Thus, in summary, to translate a time-to-corrosion minimum cover to a plan cover for a specific bridge deck one should:

(1) Assume plan cover is equal to average cover on the bridge deck.

(2) Add 0.6 inches to the minimum cover to account for the variability (standard deviation) of cover depth across the bridge deck.

(3) Apply a permeability factor of at least 1.2 to account for the laboratory to field transfer of permeability data.

(4) Use judgment to determine if additional cover is needed because the field equivalency of 830 daily laboratory salt applications is not known.

It is also worthwhile to note the minimum cover depths given are based on permeability considerations and do not take into account the structural design problems associated with their use.

Another important finding of the time-to-corrosion study is that cement factor is not an important determinant of a concrete's resistance to chloride ingress. High cement contents are often used in bridge deck construction to maintain a workable mix at a lower water-cement ratio. Although data in this report support this procedure, they also show that the reduced chloride ingress achieved is due only to the reduction in water-cement ratio and not to the increase in cement factor. Thus, other methods that allow a reduction of water-cement ratio without the addition of extra cement would be expected to perform equally well.

There has been a significant amount of discussion by others concluding that the FHWA time-to-corrosion cement factor findings differ from the findings of Stratfull in California (5,6). However, in-depth analysis indicates that there is actually very little difference in the findings but merely a difference in the method of presentation.

Stratfull used constant slump concretes of both varying water-cement ratio and varying cement factor. He found that the higher cement content and lower water-cement ratio concretes yielded greater resistance to chloride penetration. In the time-to-corrosion work, FHWA permitted slump to vary but held either water-cement ratio or cement content constant when the other parameter was varied. Thus, the two efforts used drastically different mix designs.

However, Stratfull's testing arrived at the same conclusion; that water-cement ratio had a significant effect on time to detrimental corrosion and cement factor did not, even though he did not report it as such.

Stratfull defined the following formula for determining years to deterioration of concrete exposed to saline water:

$$RT = \frac{10^{0.0442C}\, C^{0.717}\, Si^{1.22}\, 1011}{K^{0.42}\, Wm^{1.17}}$$

Where: Rt = years to deterioration of concrete exposed to saline water
C = sacks of cement per cubic yard of concrete
Si = depth of steel below the surface in inches
K = chloride concentration of water (ppm)
Wm = mixing water in percent of concrete volume

Although the formula is based on laboratory absorption data and field survey information on piling and other submerged concrete, it will be applied here to illustrate a point.

The formula uses water content (not w/c ratio but gross mix water) in the denominator and cement factor in the numerator. However, cement content divided by water content is the inverse of water-cement ratio. Thus, Stratfull's formula is considering water-cement ratio directly even if it is not shown.

If the time-to-corrosion concretes of constant w/c ratio and rebar cover but different cement factors were analyzed by the above formula, the following would result (K=18,200 ppm, Si=rebar cover=2.0 inches=51 mm):

w/c = 0.5, CF = 6 concrete - life, Rt = 9.4 years

w/c = 0.5, CF = 7 concrete - life, Rt = 9.7 years

w/c = 0.5, CF = 8 concrete - life, Rt = 10.1 years

Thus, only a 3 to 4 percent increase in life is predicted per bag of cement. This low percentage is undoubtedly less than the variability of the data used to develop the formula and thus less than the accuracy which could be expected from the formula.

However, if the varying water-cement ratio concretes with constant cement factor were used in the formula, the following would result:

w/c = 0.4, CF = 7 concrete - life, Rt = 12.7 years

w/c = 0.5, CF = 7 concrete - life, Rt = 9.7 years

w/c = 0.6, CF = 7 concrete - life, Rt = 7.9 years

Thus, the formula predicts a 30 percent increase by reducing the water-cement ratio from 0.50 to 0.40. In other words, by the formula, a drop in the water-cement ratio from 0.50 to 0.40 is 8.5 times more effective than an increase in cement factor of 1 bag/yd^3 (94 lbs/yd^3= 56 kg/m^3). If the 3 to 4 percent cement factor effect was ignored, the formula could be approximated by

$$\text{Rt approx.} = \frac{129\ Si^{1.22}}{K^{0.42}\ (w/c)}$$

where w/c is the water cement ratio by weight.

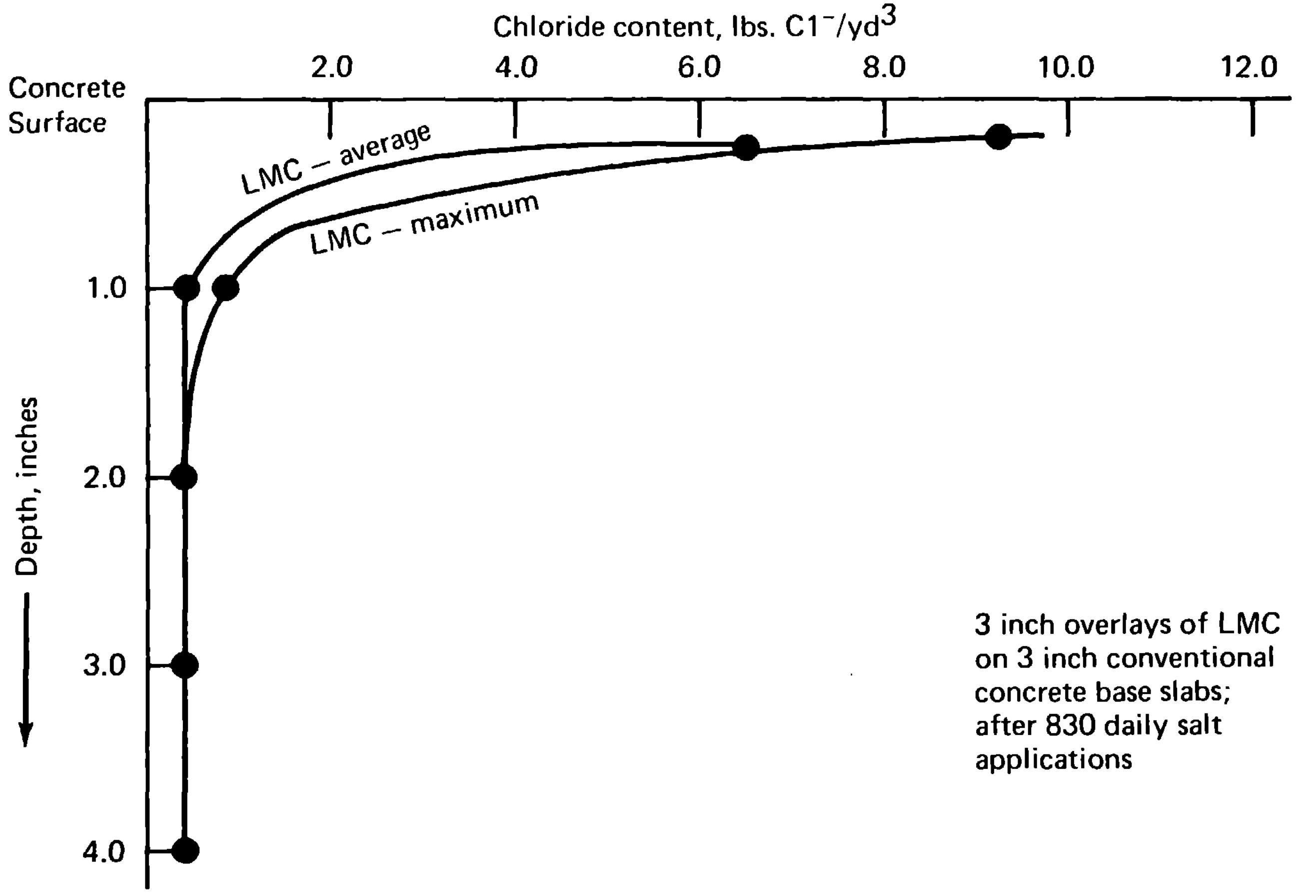
Thus, the two studies have shown the same fact; that the concrete's water-cement ratio is the primary determinate of a mix design's resistance to chloride ingress.

Latex Modified Concrete

The latex modified concrete had a cement factor of 6.75, a water-cement ratio of 0.40, and a slump of 5.0 inches (127 mm). The latex loading was 23.6 gallons of styrene butadiene emulsion per cubic yard of concrete (116.8 l/m^3). Four 20 ft^2(1.86 m^2) slabs were overlaid with 3 inches (76 mm) of the modified concrete. The cover over the reinforcing steel was 1.0 inch (25 mm).

After 830 daily salt applications no rust stains or cracks were visible on the surfaces of the slabs and no delamination could be detected. Half cell potentials were generally in the passive (<0.20 V CSE) and inconclusive (0.20 to 0.35 V CSE) ranges throughout the test period. The average and maximum chloride profiles defined using 62 samples from various depths within the material are presented in figure 11. The profiles show significant chloride in the top half inch (13 mm) but only baseline values below 1.0 inches (25 mm). The 1.0 inch (25 mm) chloride content showed no significant increase as a result of the additional 469 salt applications (361 salt application maximum 1.0 inches (25 mm) Cl^-= 0.79 lbs/yd^3= 202 ppm and 830 application maximum Cl^-= 0.86 lbs/yd^3= 220 ppm). Statistically, the depth of latex modified concrete cover required to protect 95 percent of the reinforcing steel from the penetration of sufficient chloride to cause corrosion through 830 salt applications was 0.95 inches (24 mm). Similarly, the threshold depth based on the maximum chloride value encountered was slightly under 1 inch (25 mm).

Thus, the latex modified concrete has continued to be highly superior to conventional concrete. Although the material is not impermeable, a 1 inch (25 mm) cover of latex modified concrete will supply the same protection as about 3 inches (76 mm) of conventional (w/c=0.5) bridge deck concrete.



3 inch overlays of LMC
on 3 inch conventional
concrete base slabs;
after 830 daily salt
applications

Figure 11. Effect of latex modified concrete (LMC).

Polymer Impregnated Concrete (PIC)

Four 27 inch by 32 inch by 6 inch (0.68 m by 0.81 m by 0.152 m) reinforced concrete slabs (two each with the 0.40 and the 0.50 water-cement ratio, cement factor = 7 mix designs) were totally impregnated by Brookhaven National Laboratory with methyl methacrylate and radiation polymerized. Because of the hardness of PIC, problems were encountered when a rotary hammer was used to take pulverized samples of the impregnated concretes for chloride analyses. Therefore 2 inch (51 mm) cores were taken, sliced without the aid of water, and pulverized.

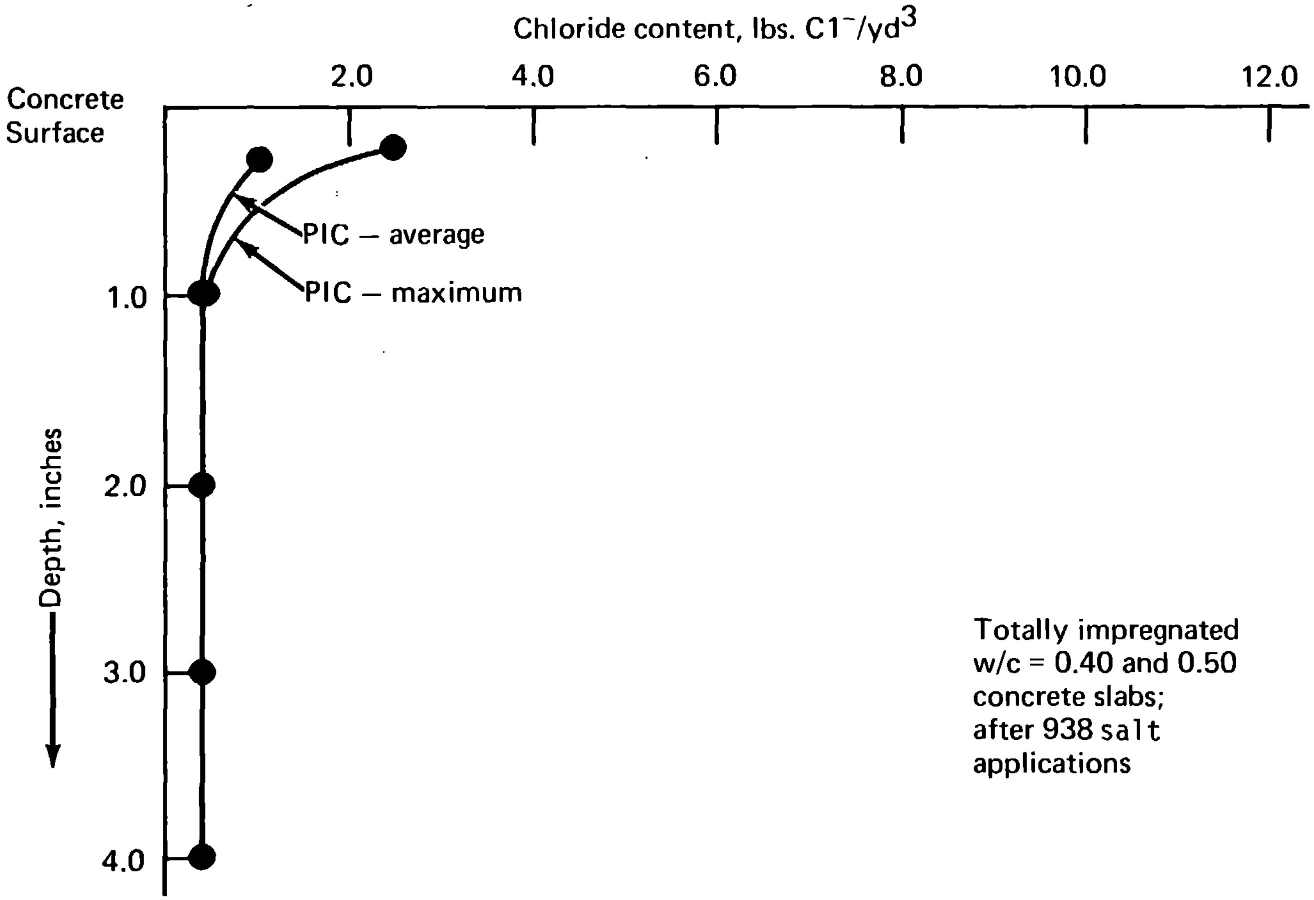
In depth analysis of the four slabs after 938 daily salt applications yielded the average and maximum chloride profiles shown in figure 12. No difference in chloride ingress was discernable due to the different concretes (0.40 and 0.50 w/c) that were impregnated and, therefore, the data on all four impregnated slabs are presented together. A slight buildup of chloride was found in the top half inch (13 mm) of the PIC (average Cl^{-}1.1 lbs/yd^3 = 281 ppm, maximum Cl^{-}= 2.5 lbs/yd^3= 639 ppm). However, only baseline chlorides were found at all other depths within the material. The maximum chloride threshold depth for the PIC was 0.7 inches (18 mm) after 938 salt applications. The threshold depth required to protect 95 percent of the steel from rebar corrosion was 0.6 inches (15 mm). Of all the mix designs, construction techniques, and special treatments studied, total polymer impregnation results in the least permeable material. About 3.5 to 4 times less rebar cover would be required to provide protection to the 0.50 water-cement ratio concrete for an equal time if it was totally impregnated.

Other testing on the totally impregnated slabs included 5-hour boiling water absorption tests (performed on slices from full depth cores) and tensile splitting tests.

The boiling water absorption tests indicated a 1 percent or less absorption in the top and bottom 1 inch (25 mm) core slices and a maximum absorption of 0.4 percent for the interior concrete. Unimpregnated concretes (water-cement ratios of 0.40 and 0.50) has maximum 5-hour boiling water absorptions of 6.4 and 7.5 percent, respectively.

The tensile splitting test results on three cores from the PIC slabs showed an average tensile splitting strength of 1580 psi (1.09×10^7 Pa) This strength is similar to that found in PIC made from the same materials but never subjected to deicers or freezing and thawing (10). These data indicate that there has been no loss of structural properties during the 3 years of daily salting and exposure to the elements.

In addition to the totally impregnated concrete slabs discussed above, four surface polymer impregnated slabs (4 ft x 2 ft x 4 inches) (1.2 m by 0.61 m by 0.10 m) are under test in this study. The slabs were prepared by the U.S. Bureau of Reclamation during development of a technique for in-situ impregnation of bridge decks, under an FHWA research contract (11), and were impregnated to a depth of about 1 inch (25 mm).



Totally impregnated
w/c = 0.40 and 0.50
concrete slabs;
after 938 salt
applications

Figure 12. Effect of total polymer impregnation (PIC).

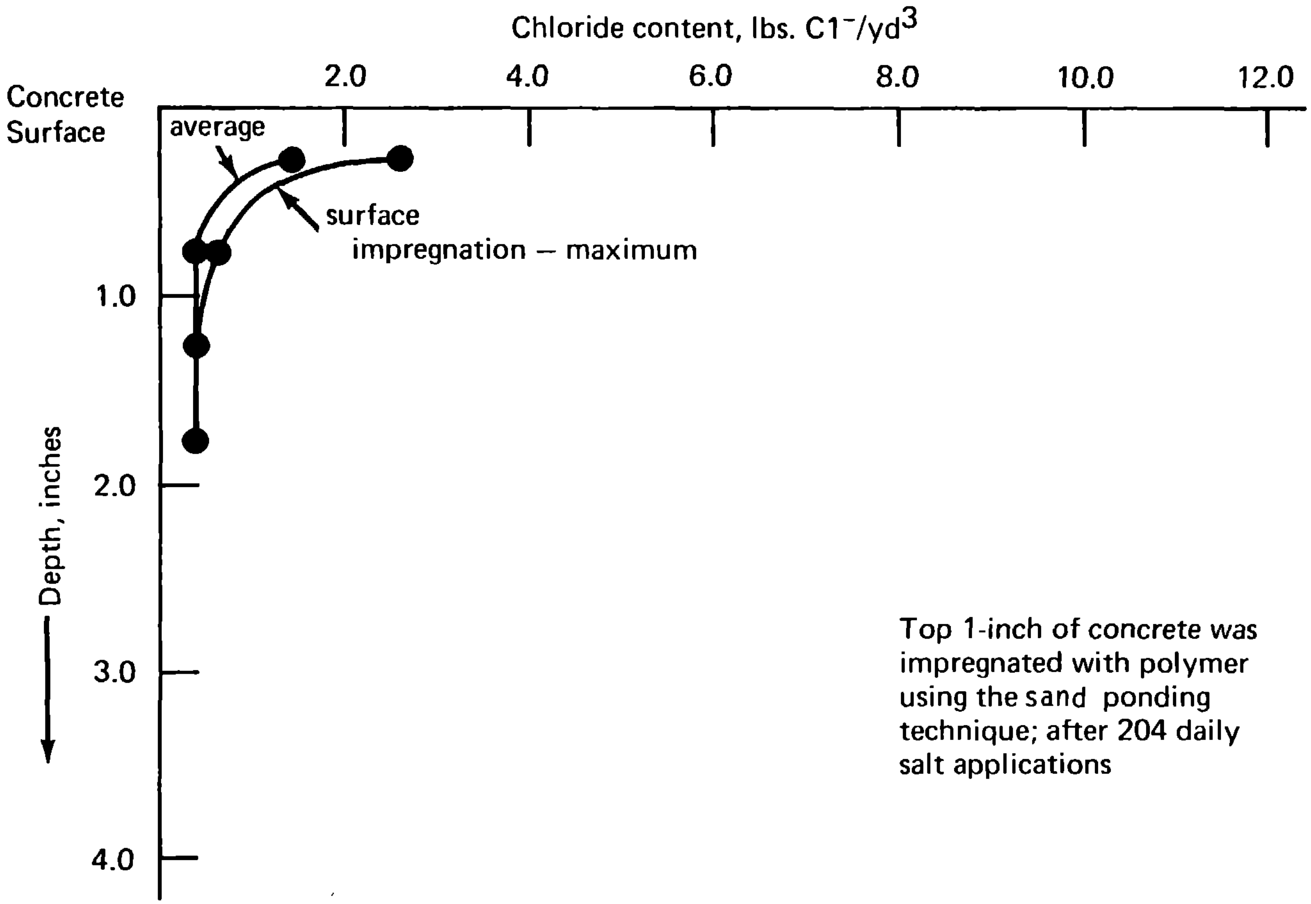
The average and maximum chloride profiles found within the surface impregnated concretes after 204 salt applications are shown in figure 13. As with the totally impregnated slabs, a slight buildup of chloride was found in the top half inch (13 mm) but no significant penetration was seen below that point. The maximum chloride threshold depth after 204 daily salt applications was only 0.6 inches (15 mm).

Although the surface impregnated concrete is performing similar to totally impregnated concrete with respect to resistance to chloride ingress, it has a significantly higher 5-hour boiling water absorption. The impregnated portion of cores taken from each of the four slabs showed absorptions of 3.7, 3.7, 3.6, and 3.0 percent. The unimpregnated underlying concrete exhibited 5-hour boiling water absorptions of 4.9 to 7.5 percent. The higher water absorptions of surface impregnated specimens, as compared to totally impregnated specimens, is believed to be related to their lower polymer contents. Total impregnation of concrete involves the application of a vacuum prior to impregnation, and pressure during impregnation, both of which increase the polymer content. In addition, when impregnation takes place from only one surface, as is done in the in-situ surface impregnation technique, the concentration of polymer will be lower, since the material which has soaked into the concrete may tend to continue to migrate to unimpregnated areas until polymerization is completed. However, since very little chloride has penetrated the surface impregnated slabs, the increased water absorption appears to be of no consequence. The difference does, however, point out the danger of using water absorption as the sole indicator of potential chloride ingress.

Follow-up chloride analyses will be performed on the surface impregnated slabs after 830 salt applications and reported at a later date.

Absorptive Form Liner and Vacuum Treatment

To define the effectiveness of absorptive form liner, four 20 ft^2 (1.86 m^2) slabs were fabricated using the 0.40 and 0.50 water-cement ratio concrete with 1 inch (25 mm) of cover over the reinforcing steel. Immediately after finishing, the absorptive form liners (0.5 inch - 13 mm - thick absorbent fiber boards) were placed on the surface and remained in place throughout the 7-day curing period. By weighing the liners periodically during set of the concrete and assuming that all water removed was extracted from the top 1 inch (25 mm) of the concrete, it was determined that the final average water-cement ratios were 0.31 for the concrete with an original w/c ratio of 0.40 and 0.36 for the concrete with an original w/c ratio of 0.50.



Top 1-inch of concrete was impregnated with polymer using the sand ponding technique; after 204 daily salt applications

Figure 13. Effect of surface polymer impregnation.

Figure 14 shows the maximum chloride profiles found after 830 salt applications within the 0.40 and 0.50 w/c slabs which were treated with absorptive form liner and the untreated control slabs with equal water cement ratios. The maximum threshold depths for the concretes were:

w/c = 0.40 control - 1.8 inch (46 mm) maximum threshold depth

w/c = 0.40 AFL - 1.8 inch (46 mm) maximum threshold depth

w/c = 0.50 control - 2.9 inch (74 mm) maximum threshold depth

w/c = 0.50 AFL - 1.9 inch (48 mm) maximum threshold depth

Thus, the data show that the use of absorptive form liner with the 3-inch (76 mm) slump typical bridge deck concrete (w/c=0.5) will improve the concrete's resistance to chloride penetration to that of a 0.40 water cement ratio concrete. In other words, AFL used on a 0.50 water cement ratio concrete will permit reduction in the depth of rebar cover of about 1 inch (25 mm) without increasing the probability of rebar corrosion.

However, the absorptive form liner had no effect on the concrete which had an initial water-cement ratio of 0.40. Thus, its use with low water cement ratio concretes is not recommended.

Four 20 ft^2 (1.86 m^2) slabs (2 each with the 0.40 and 0.50 water-cement ratio mix designs) were subjected to vacuum treatment immediately after strike-off of the concrete surface. The vacuum was applied to all 4 slabs for a period of 1 hour and the vacuum pressure was 20 inches (508 mm) of mercury (Hg) for 3 slabs but only 13 inches (330 mm) of mercury on the other slab because of equipment difficulties. The w/c=0.40 concrete slab which was treated at only 13 inches (330 mm) Hg performed very poorly with electrical potentials indicating the initiation of corrosion in only 7 salt applications. Chloride analyses after 14 salt applications confirmed that sufficient chloride was available at the rebar level (1.0 inch - 25 mm) to induce corrosion (maximum Cl^-=1.7 lbs/yd^3 = 434 ppm), and rust stains were first seen on the slab surface 5 months after testing began. Evidently, in this slab, the lower vacuum pressure was insufficient to close the continuous capillaries created by water removal and deicer solution was thus afforded a clear path to the reinforcing steel.

The other three vacuum-treated slabs, processed at a higher vacuum pressure, have not exhibited this unusually high chloride permeability. Based on the amount of water removed, the average final water-cement ratio in the slabs with an initial w/c of 0.50 was 0.41 and the final average w/c for the remaining 0.40 initial w/c slab was 0.35.

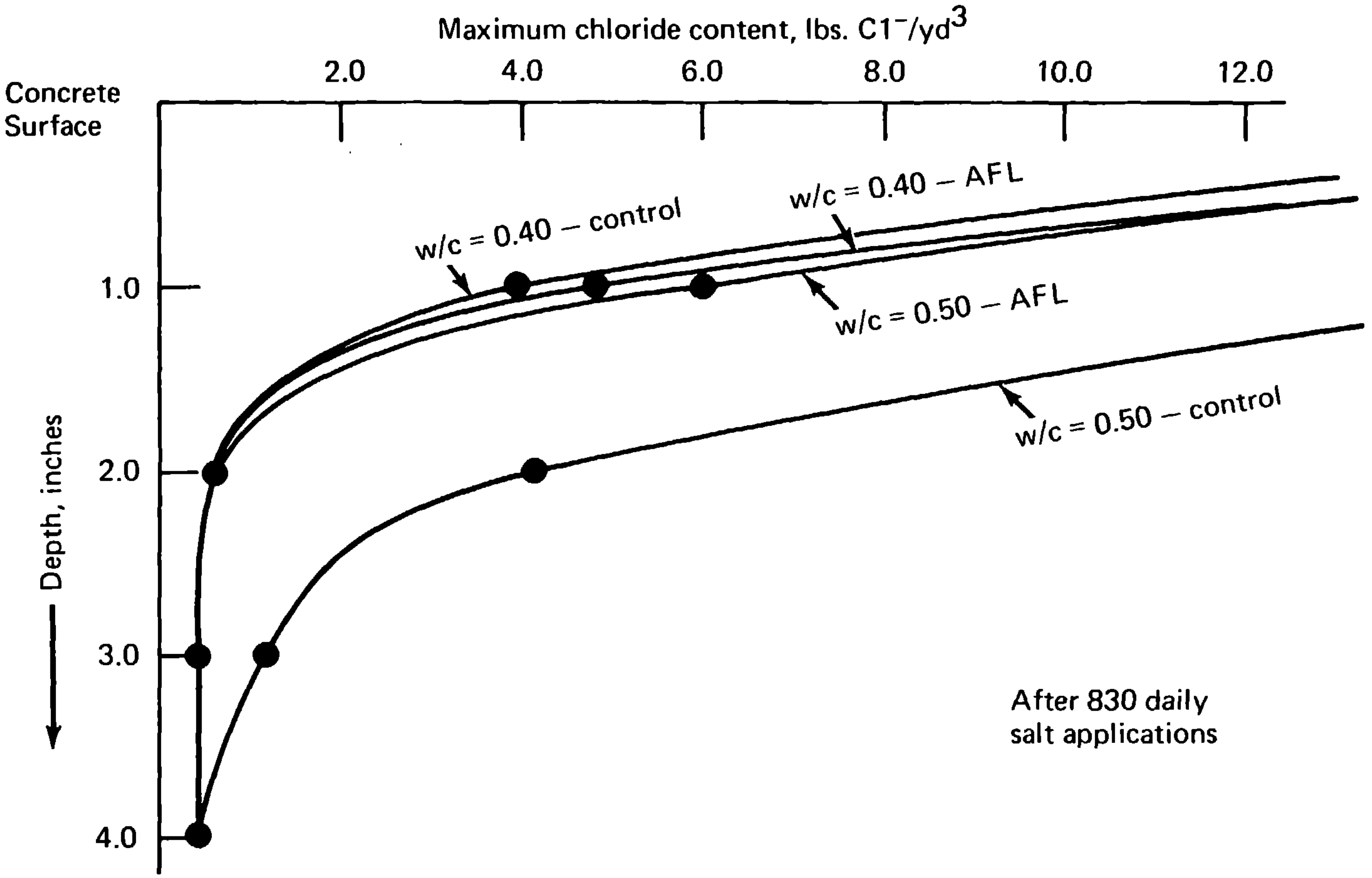


Figure 14. Effect of absorptive form liner (AFL).

Figure 15 presents the chloride profiles for the vacuum-treated and the control slabs of both water-cement ratios after 830 salt applications. The maximum chloride threshold depths were:

w/c = 0.40, control - 1.8 inch (46 mm) maximum threshold depth

w/c = 0.40, vacuum treated - 1.6 inch (41 mm) maximum threshold depth

w/c = 0.50, control - 2.9 inch (74 mm) maximum threshold depth

w/c = 0.50, vacuum treated - 1.7 inch (43 mm) maximum threshold depth

The above data show that vacuum treatment of the 3 inch (76 mm) slump typical bridge deck concrete (w/c=0.5), at a pressure of 20 inches (508 mm) of mercury, resulted in a significant reduction in chloride ingress. As with AFL, the w/c=0.50 vacuum-treated concrete is equivalent to an untreated 0.40 w/c ratio concrete. However, vacuum treatment at 20 inches (508 mm) Hg does not significantly reduce chloride ingress into a concrete which has an initial water-cement ratio of 0.40.

Thus, the time-to-corrosion data indicate that vacuum treatment and absorptive form liners provide almost identical benefits. Both techniques will reduce the chloride permeability of a 0.50 water-cement ratio concrete and both are ineffective on a concrete with an initial water-cement ratio of 0.40.

Stearate Waterproofer

A poly-oxo-aluminum stearate waterproofer was evaluated as a surface treatment. One coat of the proprietary material was brushed on a single w/c=0.50, CF=7 slab with 1 inch (25 mm) rebar cover at a rate of 1 gal/20 yd^2 (1 litre/4.4m^2).

Because the surface treatment was applied and testing was started 1 year after slab fabrication, this slab can be compared directly to the "aged" slab discussed in a previous section.

Electrical half-cell potentials have remained in the inconclusive range for both the stearate and the control slab. Chloride profiles were determined from 15 samples taken from each slab after 600 salt applications and the findings are shown in figure 16.

The data show that although a slight reduction in chloride intrusion may have been achieved as a result of the surface treatment, sufficient chloride to induce corrosion migrated to depths greater than 2 inches (51 mm) after 600 salt applications. The maximum chloride threshold depth for the stearate "waterproofed" slab was 2.7 inches (69 mm). Thus, the stearate waterproofer does not render the concrete impermeable and appears to be of little value as a method to prevent chloride induced corrosion of bridge deck reinforcing steel.

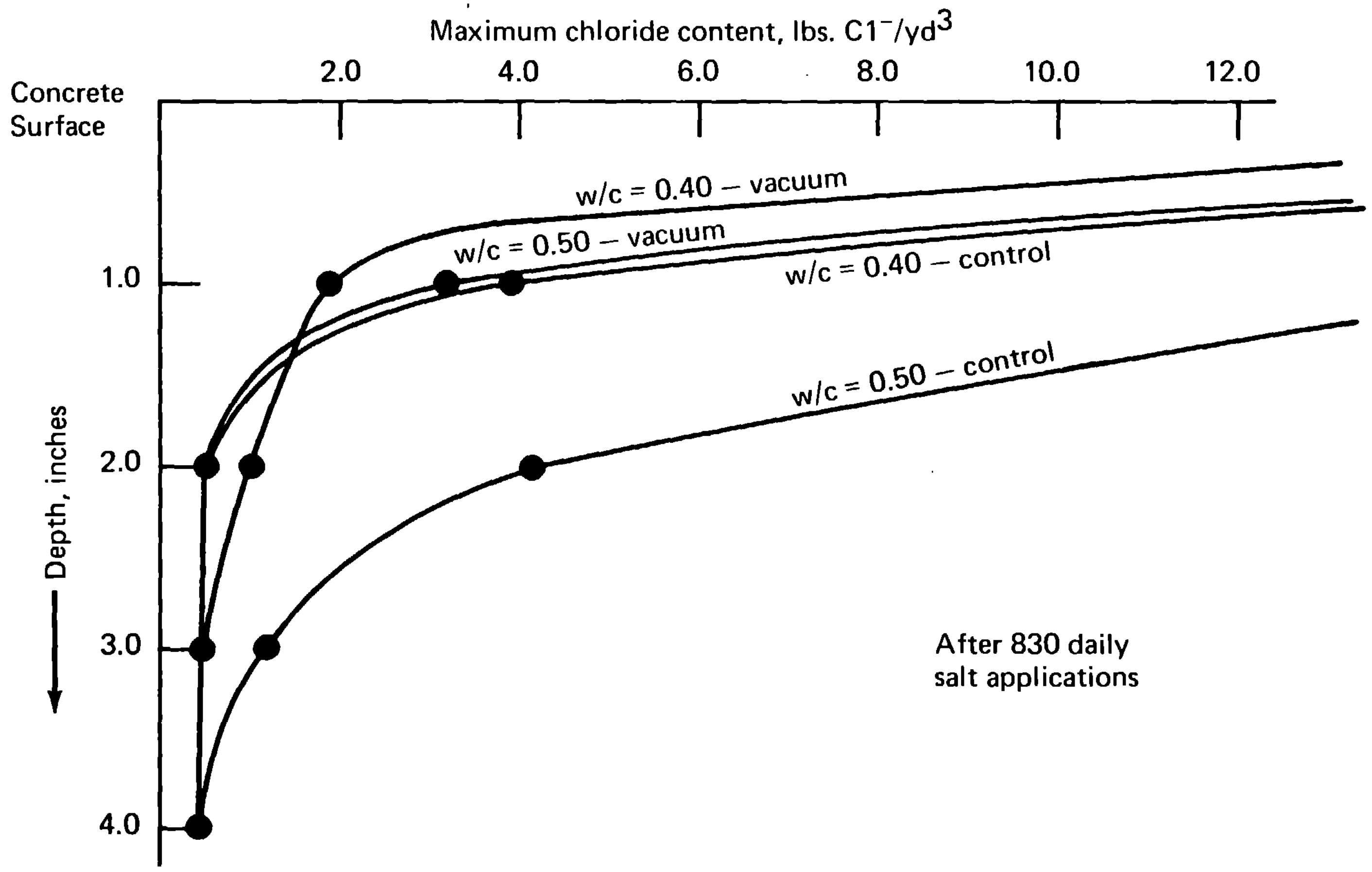


Figure 15. Effect of vacuum treatment.

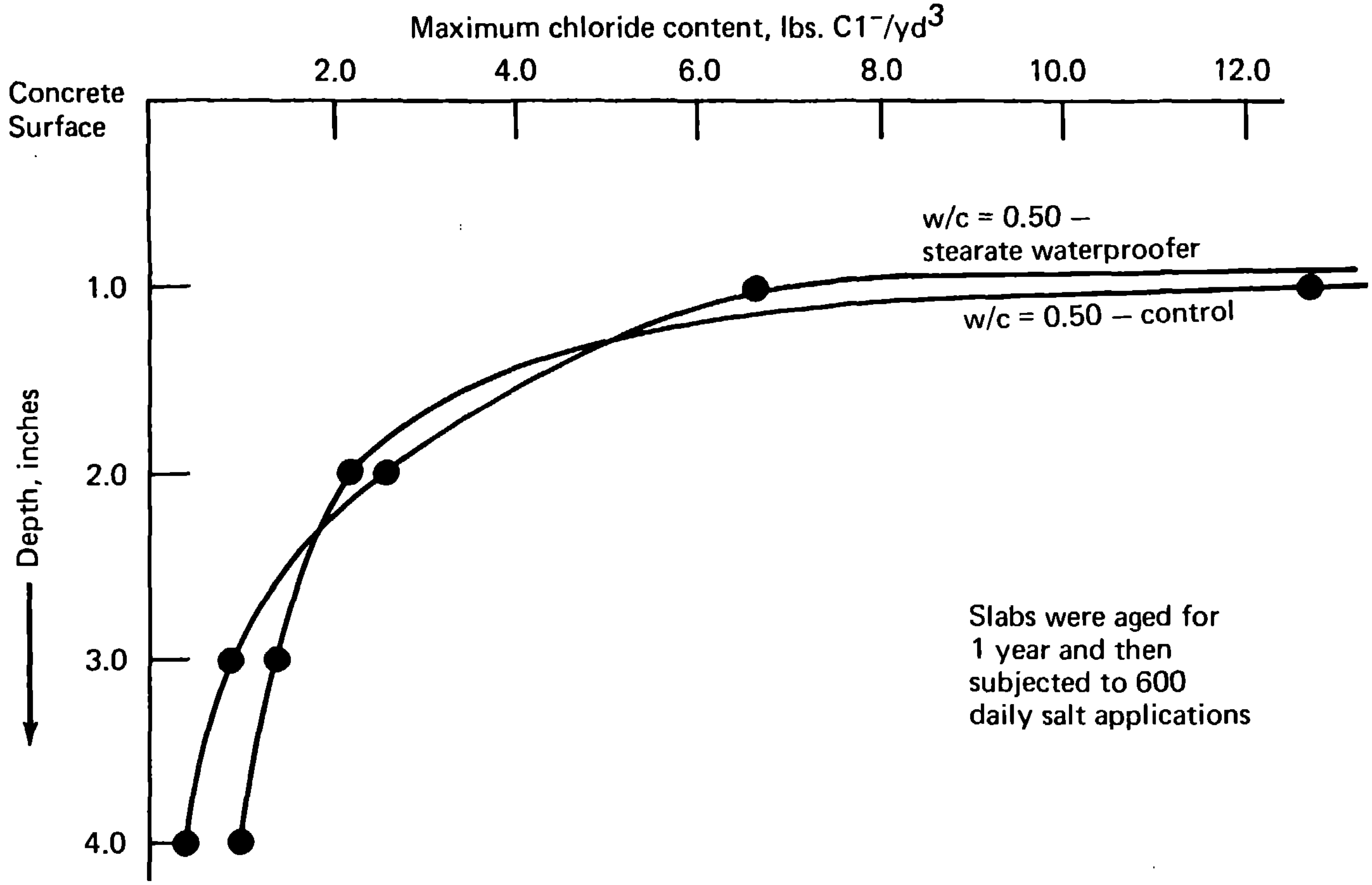


Figure 16. Effect of stearate waterproofer.

Boiled Linseed Oil Surface Treatment

Two applications of a 50/50 mixture of boiled linseed oil and kerosene were brushed on four 20 ft^2 (1.86 m^2) slabs 28 days after fabrication. The initial application was brushed on at a rate of 1 gal per 40 sq yd (1 litre/8.8m^2) and the second treatment was applied the next day at a rate of 1 gal per 67 sq yd (1 litre/14.9m^2). Two of the linseed oil-treated slabs had a water-cement ratio of 0.40 and the concrete in the other two slabs had a water-cement ratio of 0.50. A 1 inch (25 mm) concrete cover over the reinforcing steel was provided on all four slabs.

Electrical potentials have generally remained in the inconclusive range for the 0.50 w/c linseed oil-treated slabs and in the passive and inconclusive ranges for the 0.40 w/c ratio slabs. Rust stains were seen on a 0.50 water-cement ratio slab after about 500 salt applications. No rust stains have been found on the 0.40 w/c linseed oil-treated slabs. Similarly, no delaminations or cracks are present on any of the linseed oil-treated slabs.

Figure 17 shows the maximum chloride profiles encountered in the linseed oil-treated slabs and in similar untreated slabs. The maximum chloride threshold depths were:

w/c = 0.4, control - 1.8 inches (46 mm)

w/c = 0.4, boiled linseed oil - 1.0 inches (25 mm)

w/c = 0.5, control - 2.9 inches (74 mm)

w/c = 0.5, boiled linseed oil - 1.9 inches (48 mm)

The above data show that the brushed-on applications of boiled linseed oil were very effective in reducing chloride intrusion in concretes of both water cement ratios. The 0.50 w/c slabs treated with linseed oil performed about as well as the untreated 0.40 water-cement ratio concrete. Further, the 0.40 w/c linseed oil-treated slabs performed as well as the latex modified concrete slabs discussed earlier. However, there has been a 4.5 fold increase in the chloride content at the 1-inch (25 mm) depth in the 0.50 water-cement ratio linseed oil-treated slabs in the 500 saltings since the initial report. This indicates that, as was found in scaling tests, a linseed oil treatment is only effective for 2 to 3 years and would need to be repeated.

Because of several factors, the above findings cannot be translated directly to the field. The brush application procedure undoubtedly resulted in much better penetration and void filling than would have been achieved with a spraying procedure. More importantly, the slabs in this research were not subjected to traffic wear. The findings do, however, provide credibility to research in which procedures for deep impregnation of concrete with linseed oil are being investigated.

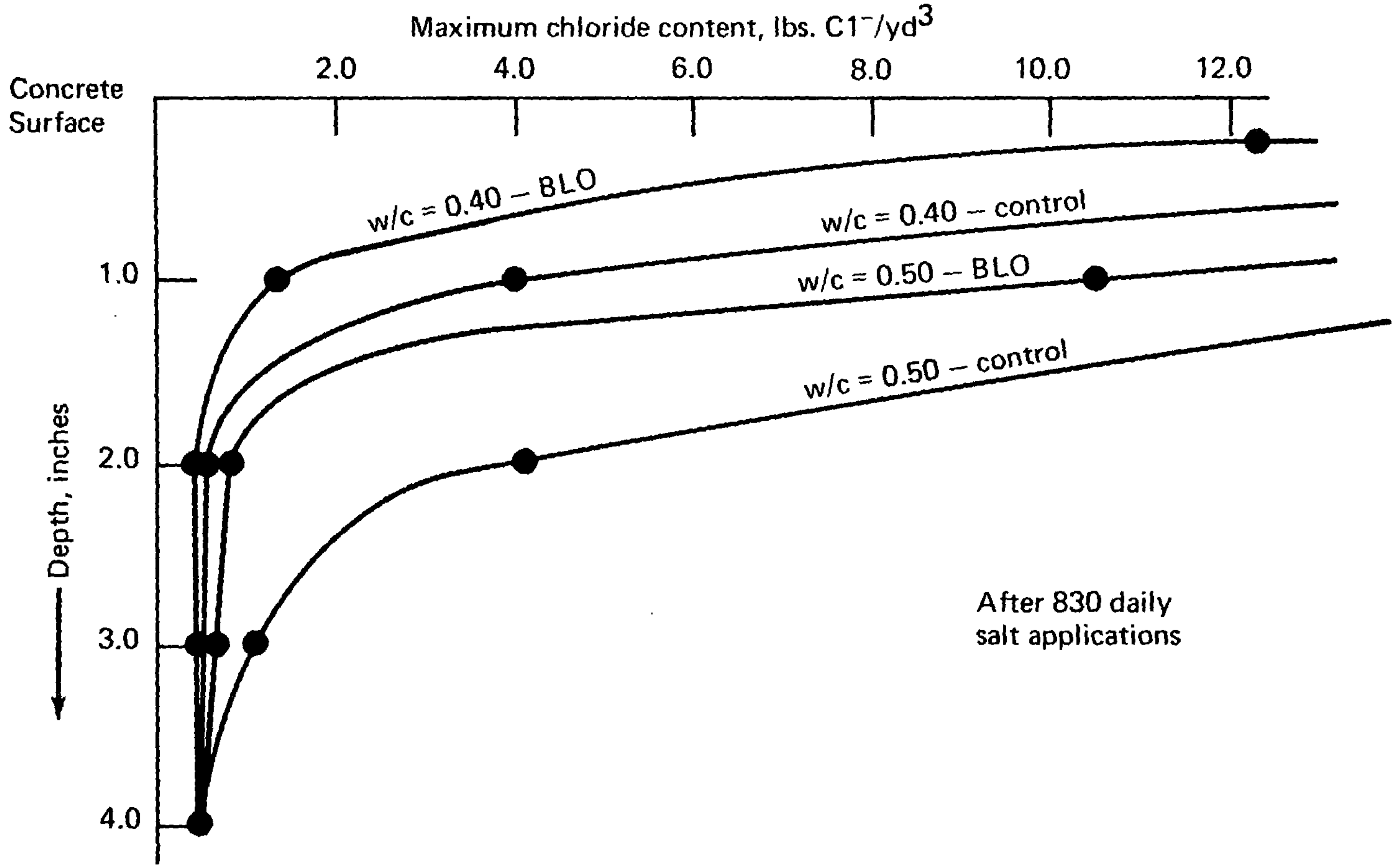


Figure 17. Effect of boiled linseed oil (BLO) surface treatment.

Permanent Steel Bridge Deck Forms

Four lightly reinforced slabs were fabricated using permanent steel bridge deck forms rather than removable plywood forms. Two slabs had a water-cement ratio of 0.40 while the concrete used in the other two slabs had a water-cement ratio of 0.50. Concrete cover over the reinforcing steel was 1.0 inches (25 mm) in all instances.

As in the case of the similar slabs with removable forms, electrical potentials indicated that corrosion was initiated in the two 0.50 water-cement ratio slabs with permanent metal forms after 7 and 14 salt applications. Rust stains on the slab surfaces confirm that corrosion is occurring. However, no cracking has occurred to date. Electrical potentials on the 0.40 water-cement ratio concrete slabs have remained in the inconclusive range throughout the 830 salt applications.

The maximum chloride profiles after 830 salt applications for the 0.40 and 0.50 water-cement ratio slabs with and without permanent steel forms are presented in figure 18. The maximum chloride threshold depths for the concrete were:

w/c = 0.40, removable forms - 1.8 inches (46 mm)

w/c = 0.40, permanent steel forms - 2.0 inches (51 mm)

w/c = 0.50, removable forms - 2.9 inches (74 mm)

w/c = 0.50, permanent steel forms - 3.0 inches (76 mm)

Although the chloride profiles show that the slabs fabricated on permanent metal forms have significantly more chloride at the depths closer to the surface, the maximum chloride threshold depths obtained using straight line interpolation for both the 0.40 and 0.50 w/c slabs with and without permanent forms are not significantly different after 830 salt applications. Therefore, since the question "To what depth is there sufficient chloride to induce corrosion?" is more important than the absolute amount of chloride at lesser depths, permanent steel forms must be judged as having no significant effect on chloride intrusion after 830 salt applications.

The condition of the steel bridge deck forms after 830 daily salt applications is also of interest. Salt solution which often ran over the small dikes on the slab surfaces during rains and down the epoxy-coated sides of the slabs onto the edges of the forms has caused severe form corrosion on each slab. A few areas along the edges (outer 6 inches) of the forms are completely disintegrated. However, the central sections of the forms which were not subjected to the salt water runoff, are in like-new condition. This finding emphasizes the need to provide adequate drainage away from the forms on bridge decks placed on permanent steel forms.

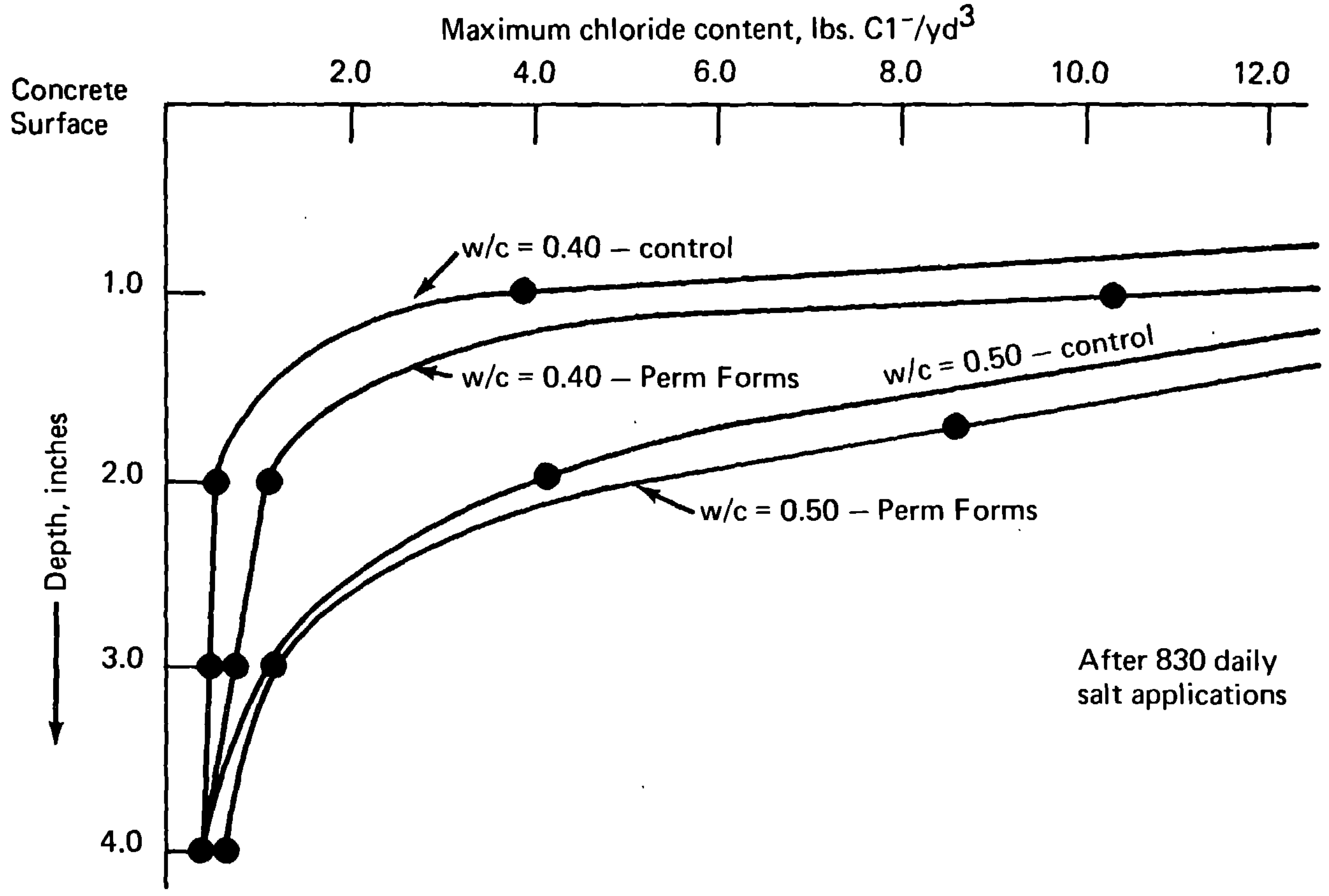


Figure 18. Effect of permanent steel bridge deck forms.

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APPENDIX

Chloride Data

CHLORIDE DATA

The chloride analyses data obtained on the various time-to-corrosion concretes and used to obtain the profile plots and the chloride threshold depths used in this report follow. Several notes are pertinent:

(1) Chloride values which were less than 0.40 lbs Cl^-/yd^3(102 ppm) are considered to be baseline values and are indicated in the table by the letters BL.

(2) The number of slabs included for each variable can be determined as follows:

(a) 4 ft x 5 ft (1.22 m x 1.52 m) slabs - divide the number of samples shown by 3 and round to the next higher integer (i.e., 17 samples/3 = 5.7 = 6 slabs)

(b) 2 ft x 2.5 ft (0.61 m x 0.76 m) slabs - 3 slabs represent each variable

(3) Mean sampling depths are shown. The actual depths were 1/16 to 0.50 inches (2 to 13 mm), 0.75 to 1.25 inches (19 to 32 mm), 1.75 to 2.25 inches (44 to 57 mm), 2.75 to 3.25 inches (70 to 83 mm), and 3.75 to 4.25 inches (95 to 108 mm).

(4) In those instances where more than 10 samples from one variable were analyzed for each depth, a statistical analysis of the data was performed, and the following additional information is included.

(a) two-sided 95 percent C.I. of mean - These two values give the 95 percent confidence interval for the mean chloride content. In other words, there is a 95 percent probability that the mean chloride content of the population falls between these two values.

(b) 95 percent level chloride - A single value that indicates that statistically 95 percent of the chlorides encountered in the population would be below this value.

(c) 95 percent chloride threshold - A depth value which was calculated identically to the maximum threshold depth procedure discussed in the body of this report except that the 95 percent level chloride contents were used.

Note: To convert inches to mm, multiply by 25.4.

To convert lbs Cl^-/yd^3 to ppm Cl^- by weight, multiply by 255.4.

Table 4. Summary of chloride analysis data.

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3		95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
w/c=0.40	0.28	17	20.0	19.4	9.02	32.5	16.6	23.5	31.0		
CF=7.0	1.0	17	1.58	1.55	BL	3.92	1.10	2.06	3.10		
conven-	2.0	17	BL	BL	BL	0.55	BL	0.46	0.52	1.7	1.8
tional	3.0	17	BL	BL	BL	0.59	BL	BL	0.51		
concrete	4.0	16	BL	BL	BL	BL	BL	BL	0.50		
830											
saltings											
w/c=0.50	0.28	18	22.1	21.4	10.6	30.2	19.4	24.8	31.1		
CF=7.0	1.0	18	11.4	11.1	5.80	17.0	9.74	13.1	16.9		
conven-	2.0	18	1.76	1.49	0.71	4.09	1.30	2.22	3.27	2.8	2.9
tional	3.0	18	0.55	0.51	BL	1.13	0.47	0.63	0.82		
concrete	4.0	18	BL	BL	BL	BL	BL	BL	BL		
830											
saltings											

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3		95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
w/c=0.60	0.28	18	27.9	24.6	18.3	60.6	22.3	33.5	46.4		
CF=7.0	1.0	18	13.7	13.1	7.85	28.6	11.2	16.2	21.9		
conven-	2.0	18	3.85	3.68	1.41	11.6	2.70	4.99	7.62	3.1	3.7
tional	3.0	18	0.77	0.64	BL	2.07	0.58	0.96	1.40		
concrete	4.0	18	0.53	BL	BL	0.97	BL	0.63	0.84		
830											
saltings											
w/c=0.50	0.28	6	24.4	21.2	17.3	44.2					
CF=6.0	1.0	6	7.65	6.53	3.96	15.0					
conven-	2.0	5	1.36	1.34	0.66	2.20					2.7
tional	3.0	6	0.47	BL	BL	0.55					
concrete	4.0	6	BL	BL	BL	BL					
830											
saltings											

Table 4. Summary of chloride analysis data (cont'd)

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
w/c=0.50	0.28	6	24.8	24.4	21.6	30.5				
CF=8.0	1.0	6	12.0	12.3	7.49	17.1				
conven-	2.0	5	1.63	1.61	0.65	2.74				2.8
tional	3.0	5	0.61	0.55	BL	0.94				
concrete	4.0	5	BL	BL	BL	0.51				
830										
saltings										
PCC Overlay										
(properly	0.28	6	15.2	15.3	9.95	18.8				
consol.)	1.0	6	1.29	0.93	BL	2.47				
w/c=0.32	2.0	6	BL	BL	BL	0.53				1.4
CF=8.75	3.0	6	BL	BL	BL	BL				
830	4.0	6	BL	BL	BL	BL				
saltings										

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3		95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
PCC Overlay											
(inadequate	0.28	6	17.9	17.8	12.2	22.2					
consol.)	1.0	6	6.25	6.11	1.96	9.75					
w/c=0.32	2.0	6	3.34	2.57	BL	7.77					3.4
CF=8.75	3.0	6	1.00	0.59	BL	2.27					
830	4.0	6	BL	BL	BL	0.51					
saltings											
Latex											
modified	0.28	13	6.52	6.71	3.57	9.24	5.35	7.68	9.68		
concrete	1.0	13	0.44	BL	BL	0.86	BL	0.52	0.65		
w/c=0.40	2.0	12	BL	BL	BL	BL	BL	BL	BL	0.95	0.96
CF=6.75	3.00	11	BL	BL	BL	BL	BL	BL	BL		
830	4.0	13	BL	BL	BL	BL	BL	BL	BL		
saltings											

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3		95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
PIC (total											
impreg.)	0.28	12	1.06	0.95	BL	2.47	0.68	1.43	2.03		
w/c=0.40	1.0	12	BL	BL	BL	0.46	BL	BL	BL		
and 0.50	2.0	12	BL	BL	BL	BL	BL	BL	BL	0.58	0.69
CF=7	3.0	12	BL	BL	BL	BL	BL	BL	BL		
938	4.0	12	BL	BL	BL	BL	BL	BL	BL		
saltings											
PIC	0.28	4	1.42	1.16	0.69	2.67					
Surface	0.75	4	BL	BL	BL	0.58					
impreg.	1.25	4	BL	BL	BL	BL					0.58
204	1.75	4	BL	BL	BL	BL					
saltings											

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
Absorptive										
form liner	0.28	5	16.9	17.6	12.6	19.0				
w/c=0.40	1.0	6	1.60	0.94	0.51	4.83				
CF=7.00	2.0	6	BL	BL	BL	0.59				1.8
concrete	3.0	6	BL	BL	BL	BL				
830	4.0	5	BL	BL	BL	BL				
saltings										
Absorptive										
form liner	0.28	6	13.5	13.1	11.2	16.9				
w/c=0.50	1.0	6	2.80	2.47	0.63	6.00				
CF=7.0	2.0	6	0.47	BL	BL	0.59				1.9
concrete	3.0	5	BL	BL	BL	BL				
830	4.0	5	BL	BL	BL	BL				
saltings										

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
Vacuum										
treated	0.28	6	15.0	14.8	10.6	19.8				
w/c=0.40	1.0	6	1.06	0.98	0.55	1.72				
CF=7.0	2.0	6	0.55	BL	BL	0.94				1.5
concrete	3.0	6	BL	BL	BL	0.47				
830	4.0	6	BL	BL	BL	BL				
saltings										
Vacuum										
treated	0.28	6	22.1	21.5	16.5	27.9				
w/c=0.50	1.0	6	1.87	1.93	BL	3.18				
CF=7.0	2.0	6	BL	BL	BL	0.47				1.7
concrete	3.0	5	BL	BL	BL	0.51				
830	4.0	6	BL	BL	BL	BL				
saltings										

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
Stearate										
water-										
proofer	0.28	3	33.0	25.9	17.9	55.1				
w/c=5.0	1.0	3	3.77	3.19	1.55	6.57				
CF=7.0	2.0	3	1.32	0.67	0.67	2.63				2.7
concrete	3.0	3	0.50	BL	BL	0.67				
600	4.0	2	BL	BL	BL	BL				
saltings										
Control for										
stearate										
slab	0.28	3	33.0	40.1	16.8	42.3				
w/c=0.50	1.0	3	8.64	11.0	2.21	12.7				
CF=7.0	2.0	3	1.64	1.53	1.27	2.11				3.2
concrete	3.0	3	0.96	0.85	0.55	1.47				
aged 1 yr.	4.0	3	0.61	0.54	BL	0.87				
before										
salting -										
600 saltings										

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
Boiled lin-										
seed oil	0.28	6	8.87	8.19	6.58	12.3				
w/c=0.40	1.0	6	0.57	BL	BL	1.29				
CF=7.0	2.0	6	BL	BL	BL	BL				1.0
concrete	3.0	6	BL	BL	BL	BL				
830	4.0	6	BL	BL	BL	BL				
saltings										
Boiled lin-										
seed oil	0.28	6	19.7	19.8	13.3	27.5				
w/c=0.50	1.0	6	7.01	6.52	4.69	10.4				
CF=7.0	2.0	6	0.54	0.51	BL	0.75				1.9
concrete	3.0	6	BL	BL	BL	0.67				
830	4.0	5	BL	BL	BL	BL				
saltings										

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
Permanent										
steel forms	0.28	6	31.8	27.5	13.0	64.8				
w/c=0.40	1.0	6	4.56	3.76	1.76	11.3				
CF=7.0	2.0	6	0.66	0.59	BL	1.09				2.0
concrete	3.0	5	0.49	BL	BL	0.70				
830	4.0	5	BL	BL	BL	BL				
saltings										
Permanent										
steel forms	0.28	6	36.7	33.0	22.4	53.9				
w/c=0.50	1.0	6	11.5	11.8	9.32	12.8				
CF=7.0	2.0	6	3.30	2.06	1.81	8.54				3.0
concrete	3.0	6	0.73	0.64	BL	1.13				
830	4.0	5	0.53	0.51	BL	0.63				
saltings										

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
2.0 x 2.5 FT SLABS										
w/c=0.40	0.28	6	15.6	14.9	13.3	17.9				
CF=7.0	1.0	6	1.23	1.26	0.79	1.62				1.3
330	2.0	6	BL	BL	BL	0.51				
saltings	3.0	6	BL	BL	BL	BL				
w/c=0.50	0.28	6	22.0	23.5	17.8	26.4				
CF=7.0	1.0	6	7.88	8.30	6.12	9.21				2.7
330	2.0	6	1.64	1.54	1.18	2.57				
saltings	3.0	6	0.64	0.62	BL	0.80				
w/c=0.50	0.28	6	2.91	2.78	1.32	5.01				
CF=7.0	1.0	6	0.85	0.88	BL	1.10				0.96
8 rock	2.0	6	BL	BL	BL	BL				
salt applications	3.0	6	BL	BL	BL	BL				

Table 4. Summary of chloride analysis data (cont'd).

Variable	Mean sampling depth, inches	Number samples	Average chloride, lbs/yd^3	Median chloride, lbs/yd^3	Minimum chloride, lbs/yd^3	Maximum chloride, lbs/yd^3	2-sided 95% C. I. of mean lbs/yd^3	95% level chloride, lbs/yd^3	95% chloride threshold depth, inches	Maximum chloride threshold depth, inches
				2.0 x 2.5 FT SLABS						
w/c=0.50	0.28	6	3.44	3.48	2.00	4.97				
CF=7.0	1.0	6	0.87	0.92	0.48	1.16				0.97
8 daily	2.0	6	BL	BL	BL	BL				
salt appli-										
cations	3.0	5	BL	BL	BL	BL				
w/c=0.50	0.28	6	7.81	7.54	6.32	10.0				
CF=7.0	1.0	6	1.60	1.55	1.05	2.15				1.5
26 winter	2.0	6	BL	BL	BL	0.49				
saltings	3.0	6	BL	BL	BL	BL				
w/c=0.50	0.28	6	8.24	8.18	5.0	11.7				
CF=7.0	1.0	6	1.15	1.21	0.85	1.72				1.3
26 daily	2.0	6	BL	BL	BL	0.43				
salt appli-										
cations	3.0	6	BL	BL	BL	BL				