Performance Evaluation of Waterproofing Membrane Protective Systems For Concrete Bridge Decks

WA-RD 61.1

August 1983

Washington State Department of Transportation
Planning, Research and Public Transportation
In Cooperation with
United States Department of Transportation
Federal Highway Administration

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
WA-RD 61.1		

4. Title and Subtitle	5. Report Date / 6. Performing Organization Code
Performance Evaluation of Waterproofing Membrane Protective Systems for Concrete Bridge Decks	5. Report Date: August, 1983 6. Performing Organization Code:
7. Author(s) Khossrow Babaei and Ronald L. Terrel	8. Performing Organization Report No.
9. Performing Organization Name and Address Washington State Transportation Center (TRAC) 121 More Hall, FX-10 University of Washington Seattle, WA 98195	10. Work Unit No. 11. Contract or Grant No.: Y-2628
12. Sponsoring Agency Name and Address Washington State Department of Transportation Highway Administration Building, KF-01 Olympia, WA 98504	13. Type of Report and Period Covered: Final Report 1/3/83 - 11/3/83 14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

This study develops a methodology for field appraisal and evaluates the effectiveness of three selected waterproofing membrane installations presently in service in the State of Washington. Based on the information obtained, none of the test installations had completely sealed the passage of salt into the concrete decks. "Active" and "uncertain" corrosion potentials existed at the rehabilitated portions; however, a decrease in corrosion activity since the time of rehabilitation was noted. Concrete deterioration after rehabilitation has occurred mainly within the boundaries of the repair work and original concrete, and it is more evident in areas with a shallower rebar depth.

It is recommended that future research include a greater number of bridge decks. In addition, service life performance data should be obtained through long-range monitoring of the installations.

17. Key Words	18. Distribution Statement
waterproofing membrane, bridge deck, reinforced concrete, salt-induced corrosion, bridge deck testing methods, bridge deck deterioration	unlimited

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
unclassified	unclassified		

Form DOT F 1700.7 (8-69)
HWY FORM 341-004 6/72

ACKNOWLEDGMENT

This work was sponsored by the Washington State Department of Transportation, in cooperation with the Federal Highway Administration, and was conducted by the Washington State Transportation Center at the University of Washington.

The authors wish to acknowledge the work and contributions of the staff of the Washington State Department of Transportation, including Ronald Schultz, Special Project Engineer, and Keith Anderson, Assistant Special Project Engineer, both of the WSDOT Materials Laboratory; and Tom Roper, Senior Associate Bridge Engineer from the WSDOT Bridge Branch.

PERFORMANCE EVALUATION OF WATERPROOFING MEMBRANE PROTECTIVE SYSTEMS FOR CONCRETE BRIDGE DECKS

by

Khossrow Babaei
Research Engineer

and

Ronald L. Terrel
Professor of Civil Engineering

Washington State Transportation Center
121 More Hall, FX-10
University of Washington
Seattle, Washington 98195

Contract No. Y-2628

Prepared for

Washington State Department of Transportation
Olympia, Washington 98504

August, 1983

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PERFORMANCE EVALUATION OF WATERPROOFING MEMBRANE PROTECTIVE SYSTEMS FOR CONCRETE BRIDGE DECKS

VOLUME I

INTERPRETATIONS, CONCLUSIONS AND RECOMMENDATIONS

I. SUMMARY

The use of salt to deice pavements contributes to the premature deterioration of reinforced concrete bridge decks. One method currently used in Washington State to combat this problem is waterproofing concrete with pavement/membrane systems. The concept is generally based on sealing the concrete deck with a membrane and covering it with an asphalt overlay in order to protect the membrane and provide a riding surface. This type of protection against salt and water intrusion is used mainly on rehabilitated bridge decks, especially when traffic volumes influence the choice of bridge deck protective systems. Wide variations in service performance of the membrane systems have been reported in the United States and generalizing their effectiveness, due to a great number of variables involved and limited data available, has been difficult. This in turn has made prediction of their service lives difficult. This study was initiated to evaluate the performance of three selected waterproofing membrane installations in the State of Washington. It was also the aim of the study to develop a methodology for field appraisal which would be used in the current work to collect data from the test sites, as well as for long range monitoring of the waterproofing membrane systems in the future.

The study was divided into three phases. During the first phase, the available background data on the candidate bridges were assembled and examined. Also, a literature survey was conducted to obtain information developed from previous investigations relating specifically to evaluation of the effectiveness of the membranes. Finally, the most appropriate test sections on each bridge deck were located and a testing plan was developed in order to give a reasonable assessment of the performance of the waterproofing installations. An interim report including the preceding activities was issued at the end of the first phase.

The second phase of the study involved field testing and data collection from the three sites in accordance with the testing plan developed in the first phase and in conjunction with the WSDOT Materials Laboratory. Also included in the second phase were necessary revisions in the testing plan based on the results of a pre-testing field trial. The data collected during this phase were tabulated, analyzed and plotted with a suitable format so that a reasonable assessment of the conditions of the membranes as well as the concrete decks could be made. The second interim report containing the foregoing activities was submitted at the end of the second phase.

In the third phase of the study, the data were carefully examined and evaluated. The findings were interpreted and final conclusions and necessary recommendations were made. In addition, guidelines for long-range monitoring of the waterproofing installations were presented in order to predict their service lives. The final report, including the overall activities conducted and information gained in the course of the study, was issued at the end of the third phase.

Several general observations and conclusions were developed. Based on the results of chloride content measurement, it was found that none of the test installations had completely sealed the passage of salt into the concrete decks. However, due to the lack of service life performance data, the actual time of breakdown and rate of salt intrusion within the lives of the membranes could not be determined. Supplementary information from a coring program supported the foregoing conclusion regarding the impermeability of the installations. Based on the results of electrical potential testing, it was found that "active" and "uncertain" corrosion potentials still existed in the rehabilitated portions. However, a decrease in corrosion activity since the time of rehabilitation was noted. No sign of corrosion or chloride content at the level of rebar in excess of the corrosion threshold value could be detected in the deck, which was waterproofed at the time of construction. The results of visual inspection and chain dragging indicated that the deterioration in the rehabilitated concrete had continued after waterproofing. The deterioration had occurred mainly within the boundaries of the repair work and original concrete, and it was more evident in areas with a shallower rebar depth.

Although the present work gives an assessment of the performance of the waterproofing systems, it is recognized that the small number of bridge decks tested herein may not allow generalizing the results obtained in this investigation for the great number of membrane waterproofed decks presently in service in the State of Washington. Thus it is suggested that future research include a greater number of bridge decks. In addition, service life performance data should be obtained through a long-range continuous monitoring of the installations. Pavement/membrane renewal on bridge decks, which is often accompanied by partial restoration of the decks, provides an excellent opportunity to begin monitoring new installations as well as rehabilitated decks. Another alternative would be to build small test sections on the decks which are not presently protected against deicing chemicals.

II. INTRODUCTION AND RESEARCH APPROACH

A. Statement of the Problem

The use of salt to deice pavements contributes to the premature deterioration of reinforced concrete bridge decks. Different methods have been developed to combat this problem. Some of these methods, such as waterproofing membrane systems, are currently being used on Washington State bridge decks. Waterproofing bridge decks with membranes is an inexpensive and expedient method of concrete deck protection against salt intrusion. The method is especially attractive when traffic volumes influence the choice of bridge deck protection systems. The concept is generally based on sealing the concrete deck (either new or rehabilitated) with a membrane and covering it with an asphalt concrete (AC) overlay in order to protect the membrane and provide a wearing surface. The system has been used in the United States, Canada and Europe. Application of the system has increased greatly in the United States since 1972.

Several studies of performance of waterproofing membrane systems have been conducted both in the laboratory and in the field in recent years (see Appendix A, Part One, Volume Two). Laboratory studies generally have included permeability tests, crack bridging, bond, and resistance to heat and impact. Nevertheless, laboratory evaluations have not been able to simulate the factors actually influencing field conditions. For example, factors such as weathering in conjunction with repeated traffic loading, substratal conditions, and workmanship are difficult to simulate in the laboratory. Studies of field performance of membrane systems have generally focused on the permeability of the systems using techniques such as cloride content measurement and electrical resistivity testing. However, some research has focused on the trend of the corrosion of reinforcement after the installation of membrane systems. The latter research applies mainly to those rehabilited bridge decks which do not require complete removal of chloride contaminated concrete for the sake of economy. These studies generally have required removal of the membrane and exposure of the concrete surface in some areas in order to conduct electrical potential testing and also to examine the extent of the concrete deterioration.

Wide variation in service performance of the membrane systems has been reported, and generalizing their performance has been difficult. While some installations have shown satisfactory waterproofing, others have shown signs of chloride penetration into the concrete decks. Few data have been available regarding the state of continuing corrosion of the reinforcing steel in rehabilitated bridge decks. Some decks, believed to be waterproofed, were found to have an increase in active corrosion after being in service for several years, while others have shown totally opposite behavior. In general, results of past surveys conducted on waterproofed decks have been conflicting, and prediction of the service lives of the systems and rehabilitated decks based on the limited data available has been difficult.

B. Objective

The objective of this study was to evaluate the performance of the waterproofing membrane systems applied on three selected bridge decks in Washington. It was also the aim of the investigation to develop a methodology for field appraisal which would be used both in the current work to collect data from the test sites and in the future for a continuing long range monitoring of waterproofing membrane installations.

C. Research Approach

1. Phase One: Data Assimilation and Experimental Design

This phase of the study (see also Part One of Volume Two) was to design a field experiment and prepare a testing plan to give a reasonable assessment of the effectiveness of the selected membrane systems applied on concrete decks. From among the many bridge decks in Washington that had already received waterproofing membrane and AC treatment, three decks were selected by the WSDOT Bridge Branch for evaluation. The decks selected depended upon previous test data available on each as well as type of restoration, geographical location, and time since the overlay was applied. The following bridges were selected:

1. Bridge 285/10 (Columbia River), near Wenatchee

2. Bridge 82/106 (Roza Canal), near Yakima

3. Bridge 5/650 (Ebey Slough), near Everett

The Columbia River bridge deck was waterproofed with a hot-applied reclaimed rubber asphalt membrane and a class B asphalt concrete (AP) mix after delaminations in the concrete were defined and repaired. The waterproofing systems on both the Roza Canal and Ebey Slough bridge decks were non-woven polypropylene fabric and asphalt emulsion applied on the deck. Class B AC mix was used for the AC overlay on both membranes. The Roza Canal bridge deck was waterproofed during its initial construction, whereas the Ebey Slough bridge deck was waterproofed during its service life after spalling concrete on the deck was repaired.

After the available data on the candidate bridges were assembled and examined (Part One, Volume Two), a literature survey was conducted to obtain information developed from previous investigations relating specifically to the methods of evaluating the effectiveness of waterproofing membrane systems and which were relative to this study. A summary of previous research is included in Part One, Volume Two, as well as the result of a computer search of the literature conducted at the WSDOT technical library regarding methods of membrane waterproofing reinforced concrete decks.

In order to minimize traffic problems, it was decided that, of the three bridges selected, only the Columbia River bridge would have a complete set of tests requiring overlay and membrane removal and replacement. The other two bridges, Roza Canal and Ebey Slough, would be inspected and tested, but without overlay and membrane removal. Since the Columbia River bridge had already been tested in detail before waterproofing and had a considerable amount of "before" condition data, it was decided to remove the membrane and AC overlay to collect similar comparable data of "after" conditions as well.

Following this step, test sections on each bridge deck were located and a plan was developed for testing the waterproofed decks. This included three test sections: 96 ft (29.1 m) long by 11.5 ft (3.5 m) wide on the Columbia River bridge deck (Figure 1), 105 ft (31.8 m) long by 18 ft (5.5 m) wide on the Roza Canal bridge deck (Figure 2), and 150 ft (45.5 m) long by 13 ft (3.9 m) wide on the Ebey Slough bridge deck (Figure 3). Several field evaluation methods were proposed in the testing plan while taking the capabilities of the WSDOT Materials Laboratory as well as the actual field conditions into consideration. These methods were:

1. Electrical resistivity of waterproofing system
2. Chain dragging AC overlay and concrete deck (if overlay removed)
3. Electrical potential of reinforcing steel
4. Core sampling AC overlay and membrane
5. Rebar depth measurement (pachometer survey)
6. Chloride content measurement

See Part One, Volume Two for detailed testing plan and sequence of testing activities on each bridge deck.

Field trips to the sites were made in order to inspect the test sections on the selected bridges before field testing. This was accomplished after a tentative test plan was developed.

2. Phase Two: Field Testing and Data Collection

Phase Two of the investigation (see also Part Two, Volume Two) involved field testing and data collection in accordance with the testing plan prepared in Phase One in conjunction with the WSDOT Materials Laboratory. Complying with the testing schedule, the actual date of testing for each bridge was coordinated with the weather. Field testing of the Roza Canal and Ebey Slough bridges was accomplished according to the initial testing plan. However, the plan for testing the Columbia River bridge, which required overlay and membrane removal and replacement, was revised due to the results of a field trial conducted in advance of testing. During the field trial, a small portion of the asphalt overlay and membrane (see Figure 1) was removed and the procedure was evaluated. The results indicated that it would be very time-consuming to remove the pavement and clean the concrete surface in all three test sections as initially planned. Thus, a revised testing plan was prepared for the Columbia River Bridge which minimized the AC overlay/membrane removal and replacement at the three test sections (see Figure 4) and disruption to traffic as well. A description of the field trial and revised testing plan are included in Part Two, Volume Two.

Generally, the testing phase followed the experimental design in every aspect of testing and no major difficulties were encountered in the field. However, there were some minor changes in the work plan due to actual field conditions. For example, the actual testing of the Ebey Slough bridge included testing an area on the bridge which was heavily patched, in addition to testing the designated test section on the deck. Included in Part Two, Volume Two is a description of different experiments and field activities conducted during the testing phase, including AC overlay and membrane removal and replacement, accomplished by the WSDOT District 2 maintenance crew.

After the raw data were collected, they were tabulated, analyzed and plotted in a suitable format to gain a reasonable assessment of the condition of both the membranes and the concrete decks. A section of Part Two, Volume Two is devoted to this topic. Also included in this section are "before" condition data (if available) arranged in the same format to make comparison easier. Although in some cases the lack of such data might make comparison impossible, the new data could be used as baseline information in the future to assess the service lives of membrane waterproofing systems as well as the rate of deterioration of the decks.

III. SUMMARY OF FINDINGS AND INTERPRETATIONS REGARDING EFFECTIVENESS OF AC OVERLAY/MEMBRANE SYSTEMS

A. Impermeability of AC Overlay/Membrane Systems

The following section presents a summary of findings and interpretations regarding the waterproofing effectiveness of the AC overlay/membrane systems. The interpretations are based mainly on the results of different evaluation methods used in this study (namely, visual inspection, electrical resistivity testing, chloride content measurement, and coring program) as well as on data obtained in previous investigations by WSDOT. Primary emphasis is placed on the ability of the systems to prevent the intrusion of salt and water.

1. Visual Inspection of Decks from Underside

Deficiencies such as wet spots and efflorescence on the undersides of decks are generally evidence of water leakage through cracks in the deck. A careful examination of the undersides of the decks in this study was not possible since they were nearly inaccessible. The limited data obtained in this regard indicates the existence of considerable transverse efflorescence under the Columbia River and Ebey Slough bridge decks. Although efflorescence is an indication of water seepage through cracks and joints, it should be kept in mind that prior to waterproofing the Columbia River and Ebey Slough bridge decks, they had been exposed to water and salt for 26 and 19 years, respectively, during which time efflorescence could have formed. This is not true, however, for the Roza Canal bridge deck, since it was waterproofed at the time the deck was constructed. Leakage and efflorescence could rarely be seen on the underside of the Roza Canal bridge deck, thus indicating only a few local areas of failure in the waterproofing system.

2. Electrical Resistivity Testing

Based on Spellman's and Stratford's suggestion (7, 16), an excellent waterproofing material should have a resistivity greater than 500,000 ohms, a poor and perforated material should have a resistivity less than 100,000 ohms, and the performance of installations with resistivities between these two limits should be considered questionable. WSDOT specifications for newly installed pavement/membrane systems (test method no. 413A) suggests that no more than 30% of the resistivity values be smaller than 250,000 ohms with no single resistivity value below 100,000 ohms. The pavement/membrane systems applied on the Ebey Slough and Roza Canal test sites did not satisfy the latter requirement (see Figure 5). About 83% and 75% of the resistivity values recorded on these two test sites were smaller than 100,000 ohms. On the Columbia River brige deck, the resistivity values were relatively higher (Figure 6). However, two out of three test sections (test sections 2 and 3) still did not satisfy WSDOT criteria (i.e., 39% and 42% of the resistivity values were smaller than 250,000 ohms, 23% and 30% were lower than 100,000 ohms). One of the test sections (test section 1), on the other hand, satisfied the first requirement (i.e., 18% of the resistivity values were less than 250,000 ohms) but failed the second requirement (i.e., only 11% of the resistivity values less than 100,000 ohms). In general, based on the results of resistivity testing, none of the test sites on the three decks completely satisfied the requirement for a satisfactory waterproofing membrane system at the time of testing.

It is important to note that variations in moisture conditions of AC overlay can influence resistivity measurements. A dry, thick and dense AC overlay might require a longer period of soaking before making resistivity measurements. As a result, higher resistivity values could be obtained, which might imply the presence of a "waterproofed" membrane. High resistivity values recorded at the Columbia River bridge deck may be, in effect, a result of this condition. Although the AC overlay was wetted repeatedly for at least one hour before taking resistivity measurements, it may be that the extremely dry condition of the AC overlay, in conjunction with a greater thickness (i.e., an average of 1.8 in, or 4.6 cm - compared with 1.5 in or 3.8 cm and 0.9 in or 2.3 cm for the Ebey Slough and Roza Canal test sections, respectively) has been the cause of the high readings. Also, it is possible that a higher density resulted from a higher traffic flow (i.e., 27,000 ADT compared with 16,800 and 6,707 ADT for the Ebey Slough and Roza Canal bridge decks, respectively), which might have caused high resistivity values. The results of an air flow test (used for pavement compaction control) on the AC overlay at the Columbia River bridge deck (test section 2), conducted by WSDOT District 5 at the time of testing the deck, showed almost no air flow in the pavement, which is an indication of a highly dense asphalt concrete. A few extremely high resistivity values recorded along the expansion dams at the Ebey Slough test site could also be an indication of this latter condition. In this case, the overlay might have become extremely dense due to the impact of traffic in the vicinity of the expansion dams. Thus it is suggested that, where most resistivity measurements are extremely high, implying the presence of a satisfactory membrane overall, longer periods of wetting be allowed.

Invalid resistivity measurements could also be obtained in the presence of excessive moisture in the pavement. This could be due to a short circuit to the reinforcing steel through the edge of the membrane adjacent to the curb, which could cause low resistivity readings. This condition might have existed for some of the resistivity values taken at the Ebey Slough test site on the second day of testing, since it had rained the night before. In this case, however, a subsequent coring program clarified the condition of the membrane and indicated a breakdown in the membrane.

Electrical resistivity testing is, in effect, a laboratory procedure that has been taken out into the field, so extra care must be taken to avoid (or, if possible, to correct) undesirable field conditions discussed earlier to obtain valid test results. One alternative for overcoming uncontrolled field conditions may be core sampling the pavement/membrane system and conducting laboratory resistivity testing on the core samples. This procedure, although somewhat laborsome, could be very efficient when appropriate field conditions are not available. It is also important to note that the field procedure of electrical resistivity testing neglects the influence of tire pressures on saturated, flexible AC overlays, which may apply large amounts of pressure on water to pass through the membrane. It is believed that this condition should be taken into consideration in both field and laboratory resistivity testing.

3. Chloride Content Measurement

Nearly all of the chloride content resamples on the Ebey Slough (1974) and Columbia River (1975) bridge decks showed signs of chloride intrusion into the concrete decks. The resampling was limited to four chloride data points on the Ebey Slough bridge deck and two on the Columbia River bridge deck (see Tables 1 and 2). On the Ebey Slough bridge deck, the chloride penetration at a depth of 0.5 in (1.27 cm) to 1.25 in (3.18 cm) in the concrete, corresponding to nine years of service, is an average of 0.23 lb/cy (0.13 kg/m^3) on the Marysville side (i.e., the average of samples 1 through 4 in Table 1) and somewhat smaller than 1.66 lb/cy (0.98 kg/m^3) on the Everett side in the vicinity of the AC patches (i.e., sample 5 in Table 1). In this case the higher chloride penetration on the Everett side of the deck may be traced to the following factors:

1. A grade of about 2.25% which could cause slippage of the membrane under traffic flow and result in membrane breakdown.

2. Thinner AC overlay (i.e., average thickness of 1.1 in or 2.8 cm compared with 1.5 in or 3.8 cm on the test section) which could result in less protection to the membrane under repeated traffic loads.

3. Thinner concrete cover (i.e., average thickness of 1.3 in or 3.3 cm compared with 2.0 in or 5.1 cm of the test section as determined by pachometer) which might have been associated with subsidence transverse cracking directly above the uppermost reinforcing steel during original placement of fresh concrete (17). This type of cracking could in turn provide ready access to salt and water in the case of membrane breakdown.

Resampling concrete for chloride content measurement on the Columbia River bridge deck indicated an average chloride penetration of 1.46 lb/cy

(0.86 kg/m^3) at a depth of 1 in (2.5 cm) to 1.5 in (3.8 cm) and corresponding to eight years of service. The reliability of these data to represent waterproofing effectiveness of the membrane is somewhat questionable since the initial chloride sampling was done before May 1975 and the deck was waterproofed in July 1976. In addition, the location of the resamples in the 1975 survey (two data points as given in Table 2) were only approximately located due to the lack of initial information regarding the exact locations of the samples. More accurate data on chloride penetration into the concrete deck of the Columbia River bridge may be found from results of a chloride content survey on the repaired concrete installed in 1976 immediately before waterproofing the deck (see Table 3). The latter data indicate that the average chloride content in the repaired concrete at a depth of 0.5 in (1.3 cm) to 1.0 in (2.5 cm) is 0.69 lb/cy (0.41 kg/m^3) (average of four samples taken from test sections 1 and 2), with a maximum of 0.98 lb/cy (0.58 kg/m^3) and a minimum of 0.42 lb/cy (0.25 kg/m^3). Assuming an original chloride content of 0.30 lb/cy (0.18 kg/m^3) (as determined for deck concrete of the Roza Canal bridge deck in the study), the average chloride intrusion into the deck and corresponding to seven years of service is 0.39 lb/cy (0.23 kg/m^3) with a maximum of 0.68 lb/cy (0.40 kg/m^3) and a minimum of 0.10 lb/cy (0.06 kg/m^3). Although, as indicated in Table 3, higher chloride contents were obtained at a greater depth in the repaired concrete, this trend is not very common for chloride-depth profiles. Thus it is speculated that the original concrete (contaminated) might have been partially included at a depth of 1.0 in (2.5 cm) to 1.5 in (3.8 cm).

The Roza Canal bridge deck provided the best condition for finding the amount of chloride intrusion into the deck, since it was protected immediately after construction. However, knowledge of the original chloride content of the deck's concrete was necessary for this purpose. This was achieved by obtaining a chloride-depth profile in the concrete and reaching depths where chloride can hardly intrude, and studying the trend of the chloride profile. A chloride depth profile at two different locations on the deck (see Figure 7 and Table 4) suggests an original chloride content of about 0.30 lb/cy (0.18 kg/m). Subtracting the original chloride content from the chloride contents obtained at a depth of 0.5 in (1.3 cm) to 1.0 in (22.5 cm) gives a chloride intrusion of about 0.83 lb/cy (0.49 kg/m^3) at a location on the shoulder area and 0.24 lb/cy (0.114 kg/m^3) at a location in the traffic lane corresponding to the above depth range in the concrete and 14 years of service. A reason behind the higher chloride content of the shoulder area may be a higher density of the AC overlay in the traffic lane in conjunction with the cross drainage of the deck, which leads the water to the curb side.

The results of the previous discussion regarding chloride intrusion into waterproofed decks is summarized in Table 5. Based on the collected data, it is apparent that the AC overlay/membrane systems tested in this study have not completely protected the concrete decks against chloride intrusion. Determining an average annual chloride intrusion by dividing the average change in chloride content by corresponding years of service of the membranes (i.e., 9, 7 and 14 years for the Ebey Slough, Columbia River and Roza Canal bridges, respectively) would not be very meaningful. The membrane failure might have occurred only during recent years, thus giving a higher annual chloride intrusion than average. It is also possible that chloride intrusion has occurred mostly during a few cold winters accompanied by heavy salt usage. It is believed that exact knowledge of the impermeability of AC overlay/membrane systems to chlorides can be obtained through continuous monitoring of chloride content at specified locations on the deck as well as specified depths in the concrete.

There is presently very little emphasis on the characteristics of AC overlays when evaluating the behavior of waterproofing membrane systems. AC overlays have mainly been considered to provide a wearing surface for the decks. AC overlays may, however, have potential to protect the decks from chloride intrusion, or at least decrease the rate of intrusion, like rigid overlays currently used for this purpose. Similar to rigid overlays, the thickness and density of AC overlays may also contribute to resistance against salt penetration. In addition, thickness and other properties of AC overlays, such as modulus of resilience, could affect the magnitude of repeated stress transferred to the membrane from traffic loads. Magnitude and frequency of repeated stress could in turn determine the service life of the membrane.

The importance of stiffness of overlays is more pronounced for saturated overlays. Overlay deflections under heavy traffic loads are related to the stiffness of asphalt concrete mix. In saturated pavements these deflections may apply pressure on water to pass through the membrane. Thus, stiffer asphalt overlays might contribute to the impermeability of the waterproofing system, whereas too-flexible mixes may jeopardize it. A system of waterproofing membrane plus a carefully designed AC overlay may become more effective than simply a waterproofing membrane and asphalt overlay that only complies with specifications for highway overlays. However, to achieve a proper design, analytical approaches to determine the magnitude of stress within the system, as well as laboratory investigations to find the degree of salt impermeability of the system, are needed. Finite element modeling techniques could be used to study the intensity of stress in the system under different field conditions. The degree of salt impermeability of the system could be found through a rapid chloride permeability test (18). This test, which is based on the migration of chloride ions through a material under an applied electrical field, can be conducted in the laboratory in the form of 4-in (10.2 cm) diameter specimens. The test would be of great value in judging chloride permeability of different types of asphalt overlay.

4. Coring Program

The most direct evaluation of waterproofing effectiveness of the installations was achieved through the coring program. Subsequent to electrical resistivity testing, core samples of AC overlay and membrane were taken at the grid points of resistivity testing by a truck-mounted, nitrogen-cooled coring rig. Signs of defects in the membrane (pinholes) and/or presence of moisture under the membrane would indicate the presence of a "permeable" membrane. However, the reverse would not necessarily imply the presence of a "waterproof" membrane.

Examination of pinholes was conducted on the cored membrane samples from the installations applied on the Ebey Slough and Roza Canal bridge decks. This type of membrane, which uses fabric in the structure, could be separated from AC core samples and examined for pinholes. However, the membrane applied on the Columbia River bridge deck (i.e., hot-applied rubberized asphalt) was an inseparable part of the AC core sample, thus making examination of pinholes nearly impossible. At both the Roza Canal and Ebey Slough test sites, about 74% of the core membrane samples showed signs of pinholes ranging from severe to slight. It is speculated that aggregate protrusion into membrane under either asphalt paving equipment loads or traffic loads (specifically in conjunction with high temperatures) is the main cause of the pinholes.

At the Ebey Slough and Roza Canal test sites, the concrete surface in the core locations immediately after removing the membrane was wet, respectively, 76% and 54% of the time, ranging from severe to slight. The shoulder areas and areas close to the curbing generally showed higher degrees and frequencies of wetness than those close to the centerline. This is probably due to the cross slope in association with a higher density of AC overlay in the traffic lane, which could lead the water to the shoulder area (curbside). At the Columbia River test site, only 7% of the core locations showed moisture under the membrane. As explained previously, this condition would not necessarily imply the presence of a waterproofing membrane, since the dryness of the concrete surface under the membrane could be a result of both the waterproofing properties of the membrane and the moisture content of the AC overlay. For example, it would be improbable to find moisture under an AC overlay/membrane system in a warm climatic condition, and which had not been exposed to rain for a long time, even if the system were permeable. Although the AC overlay in the core locations was wetted for electrical resistivity testing prior to the coring program, it is still possible that this condition could not exactly simulate an AC overlay which has become saturated under continuous rain and at the same time is subjected to heavy traffic loads.

Thus, depending solely on the coring program, it can be stated that the installations on the Ebey Slough and Roza Canal test sites fail to prevent the ingress of moisture at the present time. The actual time of the membrane failure, however, is not known and cannot be determined through this investigation. The coring program does not give any conclusive results regarding the waterproofness of the Columbia River installation.

B. Corrosion of Reinforcing Steel

In this investigation, the only technique to determine the state of corrosion activity of the reinforcing steel was electrical potential testing (half-cell). Half-cell potentials are a valid measure of the presence of corrosion activity in reinforced concrete. Nevertheless, they do not determine the rate of corrosion of the steel. In other words, they do not measure the magnitude of steel which is corroding at a given period of time. The latter becomes important when the service life of a structure is determined based on the loss of steel rather than deterioration of concrete. It is reported (19) that it is possible to have a high half-cell potential but a low rate of corrosion (slow corrosion) which could delay the occurrence of physical distress. The opposite is also true, in which a low but active half-cell potential may be associated with a high rate of corrosion (rapid corrosion). Electrical potential testing is, thus, a useful technique only for indicating the general performance of reinforcing steel in concrete decks. In the following section, findings regarding the performance of top mat reinforcing steel in concrete decks covered with membrane waterproofing systems are presented. Findings are based on results of half-cell potential measurements of this work and the measurements obtained previously by WSDOT (if available).

1. Electrical Potential Testing

It has been empirically found (17) that a half-cell potential (relative to copper/copper sulfate reference) less negative than -0.20 volts is an indication of a passive or non-corroding steel, and more negative than -0.35 volts is an indication of active or corroding steel. For half-cell potential values in the range -0.20 - -0.35 volts, the corrosion activity of the steel is uncertain. It has also been found (14) that the total chloride content corrosion threshold for a concrete deck is 0.20%, based on the cement weight assuming that 75% of the total chloride is soluble. Since the laboratory procedure of determination of the chloride content of concrete (wet chemical analysis) gives the total chloride content in the concrete (i.e., free or soluble chloride plus combined chloride), the resulting chloride content could be compared with the total chloride content threshold.

No background information was available regarding the performance of the reinforcing steel of the Roza Canal and Ebey Slough bridge decks in the past. At the Roza Canal bridge, however, it is known that there was no corrosion activity of reinforcing steel at the time of application of the membrane, since it coincided with the construction of the new deck. As indicated in Figure 8, the half-cell potentials obtained at the Roza Canal test site (during the coring program, by removing overlay and membrane at the points of measurement) are well below -0.20 volts with the highest half-cell potential of 0.103, which is an indication of no corrosion activity on the top mat reinforcing steel. The total chloride content threshold of the deck concrete is (660) x (0.20%) = 1.32 lb/cy (0.78 kg/m^3) based on the previous discussion and the cement content corresponding to the AX concrete mix used for the deck (i.e., 660 lb/cy or 389 kg/m^3). The results of a chloride content survey at seven different locations on the test site and at the level of rebar (as determined by a pachometer) indicated a maximum chloride content of 0.69 lb/cy (0.41 kg/m), which corresponded to the maximum half-cell potential of -0.103 volts. It is believed that chloride concentrations lower than threshold value at the level of rebar are the major reason for the passive state of corrosion activity. The other reason may be the lower availability of moisture and oxygen in the asphalt covered deck compared with that of a bare concrete deck.

At the Ebey Slough test site, two different patterns of performance of reinforcing steel were found through half-cell measurements during the coring program. The first pattern, or passive state of corrosion, which was more or less similar to that of the Roza Canal test site, was found at the test section. The second pattern, or nearly active state of corrosion, was found in the vicinity of the AC patches. Both patterns are illustrated in Figure 8, in which the one representing the patched area is located toward the greater values of half-cell potential. As can be seen in the figure, among four half-cell potentials measured in the patched area, one (i.e., -0.37 volts, measured directly over an AC patch knowing that no membrane or dielectric material was installed underneath) indicates an active corrosion and two (i.e., -0.212 and -0.200 volts measured in core locations and in the vicinity of the AC patches) indicate 50% possibility for corrosion. The cause of the active and corroding state of the reinforcing steel at the deteriorating area of the deck may be the ready access of salt and water to the rebar for reasons discussed in Section III-A-3 regarding chloride intrusion in that area. A chloride content survey at the level of rebar gave an average chloride content of 1.52 lb/cy (0.90 kg/m^3) for the test section and an average somewhat higher than 2.81 lb/cy (1.66 kg/m^3) for the patched area (each chloride content measurement is an average of three measurements). There is also the possibility that corrosion existed in the deteriorating area of the deck before installation of the waterproofing system and it was never arrested after application of the waterproofing system. The cause of the pre-waterproofing corrosion may be shallow rebar depth and subsidence cracks associated with shallow rebar depth, which can give ready access to salt and water.

In general, the results of electrical potential testing on the Ebey Slough test site indicate the present performance of the reinforcing steel as "passive" on the test section and "active" on the deteriorating area. Because of the lack of baseline half-cell data, it is difficult to determine the trend of behavior of reinforcing steel after application of the waterproofing system. In other words, the influence of the waterproofing system on the performance of reinforcing steel is not known. Nevertheless, it is evident from the data that the chloride content at the level of rebar exceeds the threshold value considerably in the deteriorating area and slightly in the test section (total corrosion threshold = (610 lb/cy cement) x 0.2% = 1.22 lb/cy Cl^- or 0.72 kg/m Cl^-). Therefore, it is possible that in the case of membrane breakdown and ingress of a sufficient amount of water and oxygen (oxygen in water) into the concrete, the passive state of the reinforcing steel in the test sections becomes active. The time for concrete spalling, however, would depend on different factors, among which are cover thickness and strength as well as the magnitude and frequency of traffic loads.

The Columbia River bridge deck was the only deck in the current investigation which was rehabilitated by removing and replacing the delaminated concrete when waterproofing the deck in 1976 (2). The deck was surveyed for half-cell potentials one year before the rehabilitation in 1975 (1). The results of the current half-cell survey (1983) on three test sections at the deck, as well as those obtained in 1975 at the same locations, can be found in Figures 9, 10 and 11. The 1983 survey was conducted both in the pavement removal areas, according to a grid pattern, and in core locations randomly selected in areas not designated for pavement removal. Comparison of the pre- and post-rehabilitation surveys suggests a decrease in corrosion activity at the time of testing in 1983 for both rehabilitated-waterproofed areas (i.e., test sections 1 and 2) and just-waterproofed areas (i.e., test section 3). It is reported (20) that sufficient moisture must be present simultaneously with sufficient chloride in the concrete before steel can corrode. The relative dryness of a waterproofed deck, compared with that of a bare deck, which is totally exposed to moisture, may be a reason behind the lower corrosion activity of the reinforcing steel. However, "active" and "uncertain" potential values still can be found in some areas of the deck in the 1983 survey. This condition is more evident at test section 2, at which about 8% of the potentials represent active corrosion and about 34% represent a condition of 50% probability for corrosion. Work in Ontario (19) has shown that significant reduction in corrosion activity occurs immediately after the contaminated-deteriorated concrete is removed from around the reinforcing steel, which usually has the highest corrosion activity. Actually this condition supresses corrosion activity in repaired chloride-free areas. Continuous monitoring of the performance of this type of rehabilitation (19), however, has shown that the initial reduction in corrosion activity may not be sustained and the overall level of corrosion activity could gradually increase. One factor which may directly influence the rate of increase is reported (19) to be the volume of repair work (i.e., the amount of replaced concrete). As illustrated in Figures 12 and 13, between test sections 1 and 2 at the Columbia River bridge deck, the second one has a smaller portion of the reinforcing steel surrounded by repaired concrete. This may account for the greater level of corrosion activity of the second test section.

C. Deterioration of Concrete

Visual inspection and chain dragging were the methods used to detect the deterioration (spalling and delamination) of the deck concrete. These methods were used on both asphalt covered sections (all three test bridges) and exposed concrete sections (Columbia River bridge only). Although chain dragging bare concrete decks is considered to be one of the most accurate and simplest methods to detect delaminations, its efficiency in detecting delaminations on asphalt covered decks is questionable. Thus, the efficiency of chain dragging in the latter case was also evaluated during the course of the study. In the following sections, findings regarding the soundness of the concrete decks as well as the efficiency of chain dragging AC overlays are discussed.

1. Visual Inspection and Chain Dragging AC Overlay

Unlike exposed concrete decks, visual inspection of asphalt covered decks is difficult and needs more experience. Since the deck is overlayed, the concrete defects may be undetected at the time of inspection. During visual inspection of asphalt covered test bridges, there was no major clue which would denote the conditions of the deck slabs, except for a portion of the Ebey Slough bridge deck (south end, on grade) which was patched heavily with asphalt, possibly due to spalling of the concrete slab underneath. Two factors may be the cause of the deterioration in this portion of the deck. The first factor is the active corrosion of the top mat reinforcing steel, which results in tensile forces greater than the strength of the concrete, due to corrosion products occupying considerably more volume than the steel. The second factor is the thinner concrete cover (average of 1.3 in or 3.3 cm compared with 2.0 in or 5.1 cm of the test section as determined by a pachometer) associated with the stress reversal (i.e., tension exerted by corrosion, compression exerted by traffic (20)). This condition could eventually create cracks which would propagate from the planes of weakness in the concrete (caused by trapped bleed water under the surface crust) and extend to the surface to form spalls.

Chain dragging AC overlay was initially included in the study to determine the defects under the AC overlay, including both poor bond in the interface and concrete deterioration. Nevertheless, the method could not define the poor bond in the interface as identified during the coring program in the core locations. Neither was the procedure capable of determining the delaminations in the concrete as defined by subsequent pavement removal and chain dragging bare concrete at test section 1 of the Columbia River bridge deck. However, the method detected defects under AC overlay at test section 2 of the Columbia River bridge deck as illustrated in Figure 14. The results of a subsequent visual inspection and chain dragging the exposed concrete on a portion of test section 2 suggests that the defects detected by chain dragging AC overlay could mainly be highly progressed delaminations or spalls in the concrete deck. At the Ebey Slough bridge deck, chain dragging the AC overlay detected defects under the overlay and along the expansion dams. In this case also the defects may be extensive under-surface cracks or spalling in the concrete produced by pounding of traffic loads in those areas. Recently the results of research conducted by the Ontario Ministry of Transportation (13) have shown that by chain dragging asphalt covered decks, no areas of debonding between AC overlay and concrete deck can be detected. The same study indicated that the procedure was capable of identifying concrete scales under AC overlay and about 13% of the delaminations in the concrete. It is interesting to note that the Ontario work has concluded that, among the several systems investigated, thermography has the greatest potential as a routine procedure for detecting deterioration in asphalt covered decks when rapid assessment of the overall condition of a large number of decks is needed. A thermographic survey for delamination detection has been conducted on the east portion of the Hood Canal Bridge by WSDOT. The results from this investigation indicated that the survey worked to some extent under ideal conditions for concrete delaminations. It is believed that bringing present-day equipment up to the full potential of the theory by better definition of weather conditions (13) could make delamination detection of asphalt covered decks more rapid and efficient in the future than chain dragging.

2. Visual Inspection and Chain Dragging Exposed Concrete Decks

Visual inspection and chain dragging exposed concrete was limited to pavement removal areas at the Columbia River bridge deck. Nevertheless, during the coring program, which was conducted on all three test bridges, the exposed concrete area in the core locations, although small in size, could be visually inspected and checked for defects by tapping it with a hammer. The latter procedure, however, could not completely indicate the presence of delaminations in the concrete. The reason behind this may be the adverse influence of the surrounding AC overlay on the reflected sound as well as the insufficient accuracy of the hammer procedure itself in determining delaminations.

As mentioned earlier, on the Columbia River bridge deck, three test sections were located, among which test sections 1 and 2 had been repaired by excavating the delaminations and patching prior to waterproofing, whereas the concrete of test section 3 had been sound and no repair had been needed before waterproofing. After pavement removal and sand blasting, no deterioration could be visually detected at test sections 1 and 3. However, at test section 2, three spalled areas existed near the boundaries of the original and repaired concrete (see Figure 13). Delaminations were detected only at test sections 1 and 2 by chain dragging the exposed concrete. These delaminations are illustrated in Figures 12 and 13. As indicated in the figures, the deterioriation is almost always in the boundaries of the original and repaired concrete. The delaminations account for approximately 8% and 10% of the exposed area for test sections 1 and 2, respectively (including the spalling areas for test section 2). As discussed earlier, the defects under the AC overlay on test section 2 and at the area designated for pavement removal as detected by chain dragging AC overlay (see Figure 14) is approximately 9%. Nevertheless, the patterns of the defects in Figures 13 and 14 are not completely the same. This discrepancy may be due to possible errors in marking and mapping these areas.

An overall pachometer survey conducted on the exposed concrete areas gave the average depth of cover as 1.5 in (3.8 cm), 1.2 in (3.0 cm) and 2.4 in (6.1 cm) at test sections 1, 2 and 3 respectively. It can be seen that the intensity of the concrete deterioration at the three test sections relates indirectly to the magnitude of cover depth. It is also interesting to note that the test section with the highest intensity of corrosion at the present time shows also the highest level of deterioration (i.e., test section 2). As discussed earlier, the concrete cover in the presence of corrosion of top mat reinforcing steel and repeated heavy traffic loads may be subject to stress reversal, thus permitting initiation and propagation of cracks from the planes of weakness under the surface crust which would eventually reach the surface and form spalls.

Concrete deck repair procedure is another factor which may cause deterioration after rehabilitation. A NCHRP project entitled "Evaluation of Methods of Replacement of Deteriorated Concrete in Structures" (21) has reported the effects of repair methods on concrete deterioration as follows:

> "... many repair failures were occasioned by the removal of too little rather than too much concrete. However, complete removal was most effectively done using techniques and equipment that offered the maximum protection to areas of sound concrete on the perimeter of the patch. Where unnecessarily heavy equipment was used, it sometimes damaged the surrounding concrete and created additional areas of potential failure."

> "Saw cutting around the perimeter of the patch area to a 1 or 1.5 in depth ... appeared to contribute to the success of the repair in several ways. The saw cut provided a clean, vertical face against which the patching material could be placed for maximum contact and bond, free of feather edges. A simple technique that provided a slight "keying" action for the patches involved tilting the saw blade (by riding one wheel on a plank) to undercut the edge of the repair area at a slight angle. This helped to lock the patch in place and supplemented the bonding strength along its edge."

From the above discussion, and the presence of deterioration around the perimeter of the repaired areas, it is believed that the repair procedure is as important as protection against salt and water intrusion. The potential damage to the surrounding concrete at the time of repair, as well as the bond between the repaired and original concrete, should be given special consideration in repair procedures. The importance of the bond is more pronounced when the boundaries of the repaired areas coincide with wheelpaths or areas of stress concentration.

Visual inspection and tapping the concrete surface with a hammer in the core locations did not detect any deterioration at the Columbia River and Roza Canal bridge decks. However, at the Ebey Slough test site, where the AC patches are located, and in one core location (one out of four), delaminations could be detected by using a hammer. The success of the method in this case may be related to the highly progressed nature of the delamination in that area. At the Ebey Slough bridge deck and on the test section, two core locations (2 out of 17) showed surface distress in the concrete which looked more like scaling (freeze-thaw related distress) rather than spalling. Asphalt covered decks could have higher freeze-thaw cycles than bare concrete decks due to the heat-absorbing nature of the asphalt overlay. This condition, in conjunction with a failure of the membrane as well as lack of air entrainment of the concrete on some occasions, could result in scaling of the surface concrete.

IV. CONCLUSIONS AND RECOMMENDATIONS

A. Conclusions

The following conclusions regarding both impermeability of the waterproofing pavement/membrane installations and the conditions of the waterproofed reinforced concrete decks appear warranted from the foregoing discussion.

Impermeability of Waterproofing Pavement/Membrane Systems

1. Using chloride content measurement as the major test for waterproofing ability, none of the three test installations has completely sealed the passage of salt into the concrete decks. However, due to a lack of service life performance data, actual time of breakdown and rate of salt intrusion within the lives of the membranes cannot be determined.

2. Observations during the coring program and evidence of moisture between membrane and concrete deck indicates that two of the three test installations (Ebey Slough and Roza Canal) definitely are definitely not capable of waterproofing the decks at the present time.

3. Pinholes were detected visually in two of the membranes (fabric membrane at Ebey Slough and Roza Canal decks) during the coring program. The nature of the third installation (rubberized asphalt membrane at Columbia River deck) did not permit visual detection of pinholes.

Corrosion of Reinforcing Steel

Based on the results of electrical potential testing and the criteria used for corrosion and chloride content corrosion threshold in this study, the following conclusions regarding the performance of the reinforcing steel appear warranted.

4. "Active" and "uncertain" corrosion potentials exist at the deteriorating portion of the Ebey Slough bridge deck as well as the rehabilitated portions of the Columbia River bridge deck.

5. No signs of corrosion of steel could be detected at the Roza Canal bridge deck. The main reason may be the chloride content at the level of rebar (i.e., at an average depth of 1.7 in or 4.3 cm in the concrete), which is well below the corrosion threshold value and is only slightly above the original chloride content of the concrete in some areas.

6. Comparison of half-cell potentials measured at the Columbia River bridge deck before rehabilitating and waterproofing the deck and in the current study (7 years after rehabilitation and waterproofing) indicates a decrease in corrosion activity of the reinforcing steel.

Deterioration of Concrete

7. Results of visual inspection and chain dragging exposed concrete areas indicate that the deterioration of concrete has continued at the rehabilitated portions of the Columbia River bridge deck. The

deterioration has occurred mainly within the boundaries of the repair work and original concrete, and it is more evident in the section with a shallower rebar depth.

8. The previously sound and non-rehabilitated section at the Columbia River bridge deck still remains sound at the present time. This area coincides with a greater rebar depth.

B. Recommendations

Based on the foregoing discussion, the following recommendations seem to be beneficial for improvement and enhancement of future work.

1. The current study gives an initial assessment of the waterproofing systems tested. It is recognized, however, that the small number of membrane waterproofed bridge decks tested here may not allow generalizing the results obtained in this investigation for the great number of membrane waterproofed decks presently in service in the State of Washington. In addition, service life performance data, which can be obtained only through long-range, continuous monitoring of the conditions of the systems and sites, are required to determine the accurate service lives of the installations.

2. Presently, AC overlay is considered mainly as a provider of riding surface on the decks, as well as a means of protection of the membrane. AC overlay may, however, have a potential to protect the decks from chloride intrusion. Similar to rigid overlays, the thickness, density and modulus of resilience of the AC overlays may also contribute to the resistance of the system against salt and water penetration. Thus, a system of waterproofing membrane plus a carefully designed AC overlay may become more effective than simply a waterproofing membrane and an AC overlay which ony complies with the specifications for highway overlays.

3. The procedure for concrete deck repair is an additional factor which may contribute to deterioration after rehabilitation. It is believed that repair procedures are as important as protection against salt and water intrusion. Potential damage to surrounding concrete at the time of repair as well as the bond between the repaired and original concrete should be given special consideration in repair procedures.

4. Variation in the moisture condition of AC overlay can influence resistivity measurements. Thus, extra care must be taken to avoid (or if possible to correct) such undesirable field conditions as too-wet (or too-dry) pavements. An alternative to overcome uncontrolled field conditions may be core sampling the pavement/membrane system and conducting laboratory resistivity testing on the core samples. It is also important to note that the field procedure of electrical resistivity testing neglects the possible influence of heavy traffic loads on saturated, flexible AC overlays which may apply great pressure on the water to pass through the membrane. It is believed that this condition should be taken into consideration in both field and laboratory resistivity testing.

5. Chain dragging asphalt covered concrete decks in order to detect the concrete delamination does not appear to be able to reveal the delamination completely. Thermography is reported (13) to have the

greatest potential to be developed for this purpose. The procedure is better suited to the assessment of a large number of decks when a time factor is involved. It is believed that bringing present-day equipment up to the full potential of the theory could make delamination detection of asphalt covered decks more accurate and economical than chain dragging.

V. SUGGESTED FUTURE RESEARCH

The current study gives an assessment of the effectiveness of the tested waterproofing membranes as well as the conditions of the protected reinforced concrete decks at the present time. The study also employs different field appraisal techniques and develops a methodology in order to assess the extent of the problem. However, the question of "how fast the membrane systems wear out under a given field condition and how the aging affects their performance regarding protection of the decks from corrosion and deterioration" still needs to be answered. The answers to the foregoing question, which would predict the service lives of the installations, could be obtained through a long-range monitoring of the performance of newly installed membranes under a variety of field conditions.

Based on the preceding discussion, it is apparent that the effectiveness of a waterproofing installation should be expressed in terms of its service life and the service life is, in effect, a function of the field conditions under which the membrane performs. Some of the major factors governing field conditions are: climate, magnitude and frequency of traffic loads, properties of AC overlay, geometry of deck and surface condition of concrete. A carefully designed long-range experiment would enable the engineer to use the collected data on the effectiveness of an installation to predict its service life for its own field conditions.

Since membrane waterproofing systems are generally used for rehabilitation projects, it is important that future research be focused on this particular area. The time for AC overlay/menbrane renewal on bridge decks, which is usually accompanied with partial restoration of the concrete decks, gives an excellent opportunity to begin monitoring the new waterproofing installations as well as the rehabilitated concrete decks. Service lives of partially restored membrane waterproofed decks can then be predicted and related to influencing field condition factors. The following could be used as guidelines in the future to determine the trend of the effectiveness of membrane waterproofing installations regarding the behavior of rehabilitated reinforced concrete bridge decks.

As pursued in the present study, the effectiveness of the membrane waterproofing systems should be looked at in three different directions at each time during their service lives. These are: impermeability of the AC overlay/membrane system, corrosion of reinforcing steel, and deterioration of concrete. Relevant field appraisal techniques as employed in the current investigation could then reveal the conditions of the membranes, reinforcing steel and concrete. It is important that this condition determination include the periods of time most critical regarding the performance of the reinforced concrete, such as immediately before and after rehabilitation. Next, field condition determination should be continued periodically after waterproofing, such as once or twice a year. A preliminary investigation of the conditions of the concrete deck immediately after removal of the old AC overlay/membrane system should lead to the designation of the most proper test sections on the deck. Among the factors influencing the choice of test sections are the extent of contamination, corrosion and deterioration as well as the depth of concrete cover. It will also be important that the potential damage from repair procedures be taken into consideration. This could be achieved by detecting potential deterioration on surrounding concrete immediately after excavating the deteriorated concrete. It is also essential that immediately after restoration the baseline level of contamination of the original concrete, as well as the original contamination of the repaired concrete, be obtained. The existence of such data would be crucial in order to evaluate the salt-impermeability of the waterproofing membranes. During the continuous monitoring and data collection, the trend of the impermeability, corrosion and deterioration could be determined over time. The availability of such data during the early years of service could make prediction of impermeability, corrosion and deterioration posible at any time within the life of the installation. By being able to predict the conditions of the deck in the coming years and by having condition criteria under which deck replacement would be inevitable, one can determine the useful service lives of the tested waterproofing membrane systems. Accurate analysis of the data should define the influence of each field condition factor, or variable, on the effectiveness of the installations. At this time, in the presence of an adequate amount of data, the service life determination could be generalized to include any field condition actually encountered during rehabilitation projects.

Another alternative to testing the bridge decks at the time of renewal of old pavements is to build test sections on the decks which are not presently protected against deicing chemicals. The latter alternative provides a better opportunity to bring field variables under control through the freedom of site selection. Thus, a better analysis of the data could be possible in order to define the influence of each variable on the behavior of the installations and the decks. As a guideline, Table 6 presents a possible range of variables that might be considered for a long-range evaluation program of this type. Assuming that at least eight bridges could be included in the study, each of the major variables could be isolated to determine their relative effect on performance. Thus, the sites should be selected carefully to provide a measure of the influence of the major variables at two levels (namely, "high" and "low") while the other variables are the same (namely, "moderate"). As a preliminary suggestion, the sites should have the following characteristics to represent moderate conditions:

Climate: Winter temperatures around 25 degrees F (-4 degrees C), summer temperatures around 80 degrees F (27 degrees C)

Traffic level: Average daily traffic of about 10,000

Overlay thickness: 1.5 in (3.8 cm) of asphalt concrete

Grade: About 1%

If the effects of any variable other than those shown in Table 6 are required, the number of the sites should be increased accordingly. If not, the sites should be selected with these variables in common. Each installation should have a full width of the bridge deck and be at least 100 ft long. Thus, each deck can accommodate more than one type of installation. The conditions of each test site, such as contamination, corrosion, deterioration and rebar depth should be determined in detail prior to waterproofing. The sites should then be rehabilitated according to current specifications for rehabilitation and membrane waterproofing. Detailed conditions of the test sites should be obtained once more, immediately after rehabilitation but prior to membrane waterproofing. Testing the installations and sites should be conducted continuously and the data should be studied as discussed earlier in this section.

Methodology -- based on the information obtained during the course of this study, it is suggested that condition determination, including impermeability of pavement/membrane system, corrosion of reinforcing steel, and deterioration of concrete be accomplished through the following field appraisal methods:

Impermeability of Installations. Chloride content measurement at specified depths and areas should be used as the major test to define impermeability. A supplementary visual inspection could be conducted through coring the AC overlay/membrane system. Electrical resistivity testing should not be used as the sole method to define impermeability until more knowledge of the testing is obtained and necessary refinements are made.

Corrosion of Steel. Electrical potential testing is recommended on both pavement/membrane removal areas (during rehabilitation) as well as AC overlay/membrane core locations (after rehabilitation and waterproofing). If possible, a permanent connection should be established to the rebar to facilitate future measurements.

Deterioration of Concrete: Chain dragging and visual inspection should be used for this purpose until other methods such as thermography are fully developed. Chain dragging exposed concrete should be conducted during rehabilitation. In addition, measurement of rebar depth (pachometer survey) at this stage is needed to determine if the deterioration is associated with the concrete covers. After rehabilitation and waterproofing, AC overlay can be chain dragged for determination of concrete deterioration. Additionally, the exposed concrete in AC overlay/membrane core locations can be visually inspected and tapped with a hammer for possible deterioration.

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LIST OF FIGURES

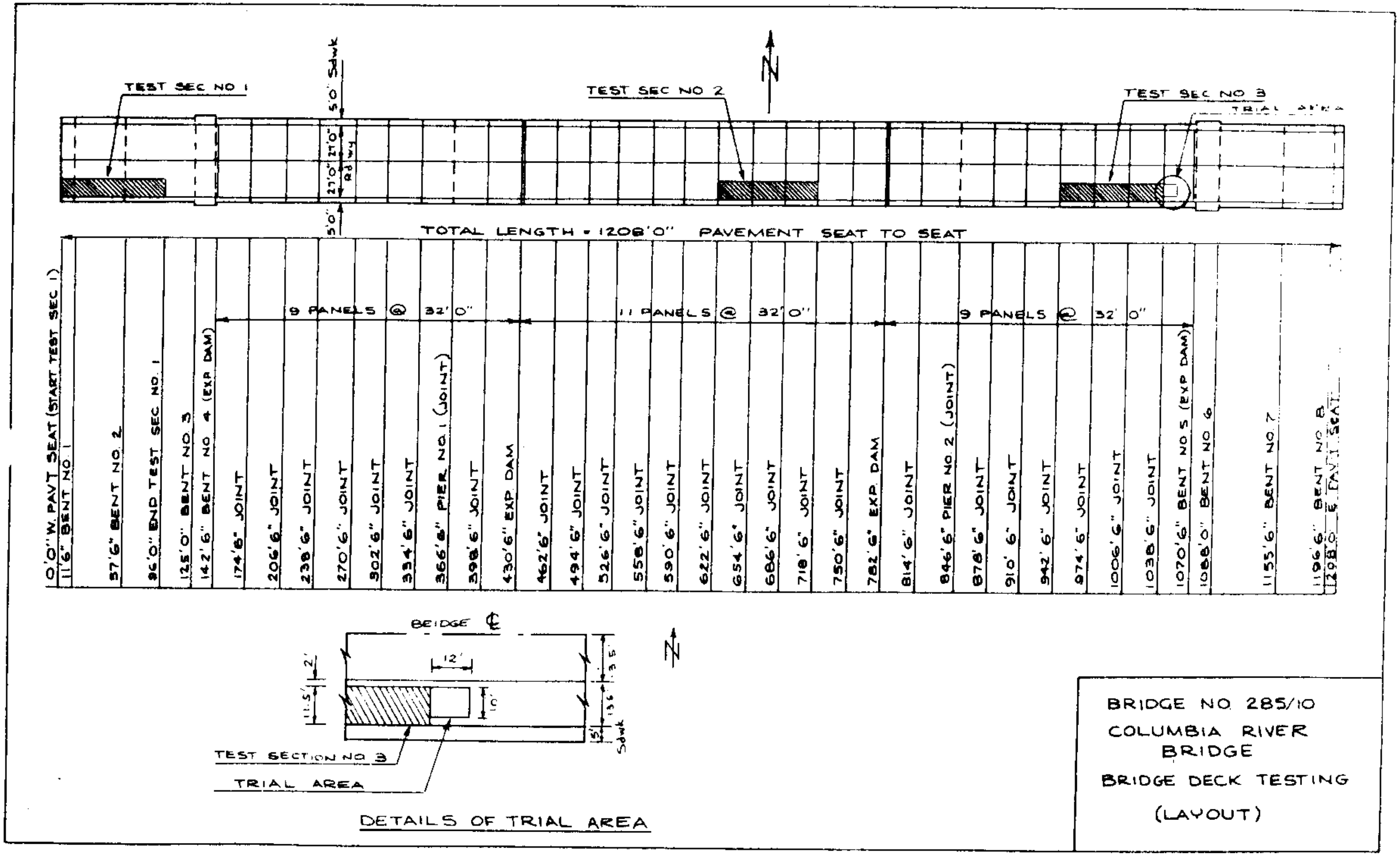

Figure 1. Layout of Initial Test Sections on Bridge 285/10 and Location of Trial Pavement Removal

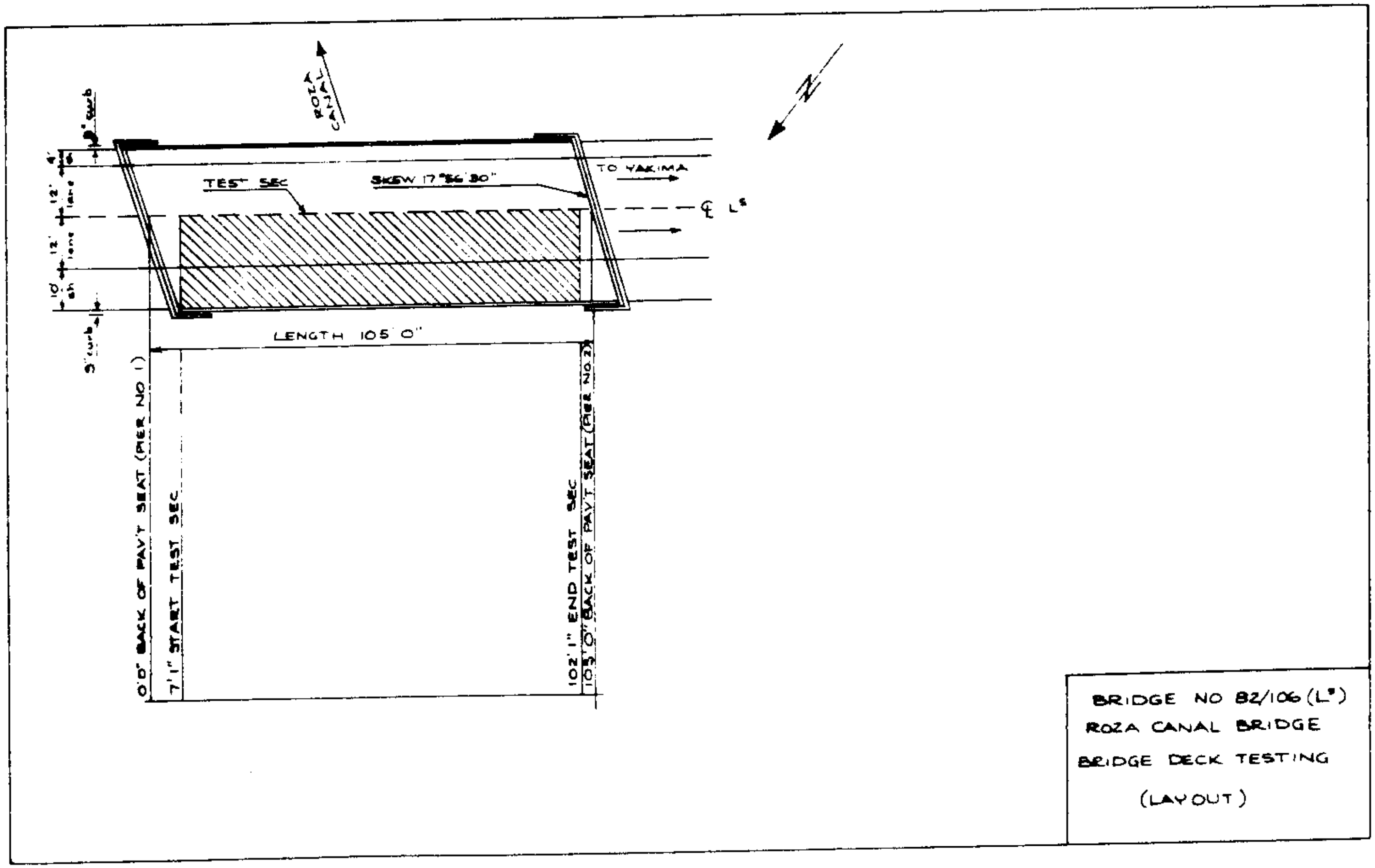

Figure 2. Test Section on Bridge 82/106 L/S (Roza Canal)

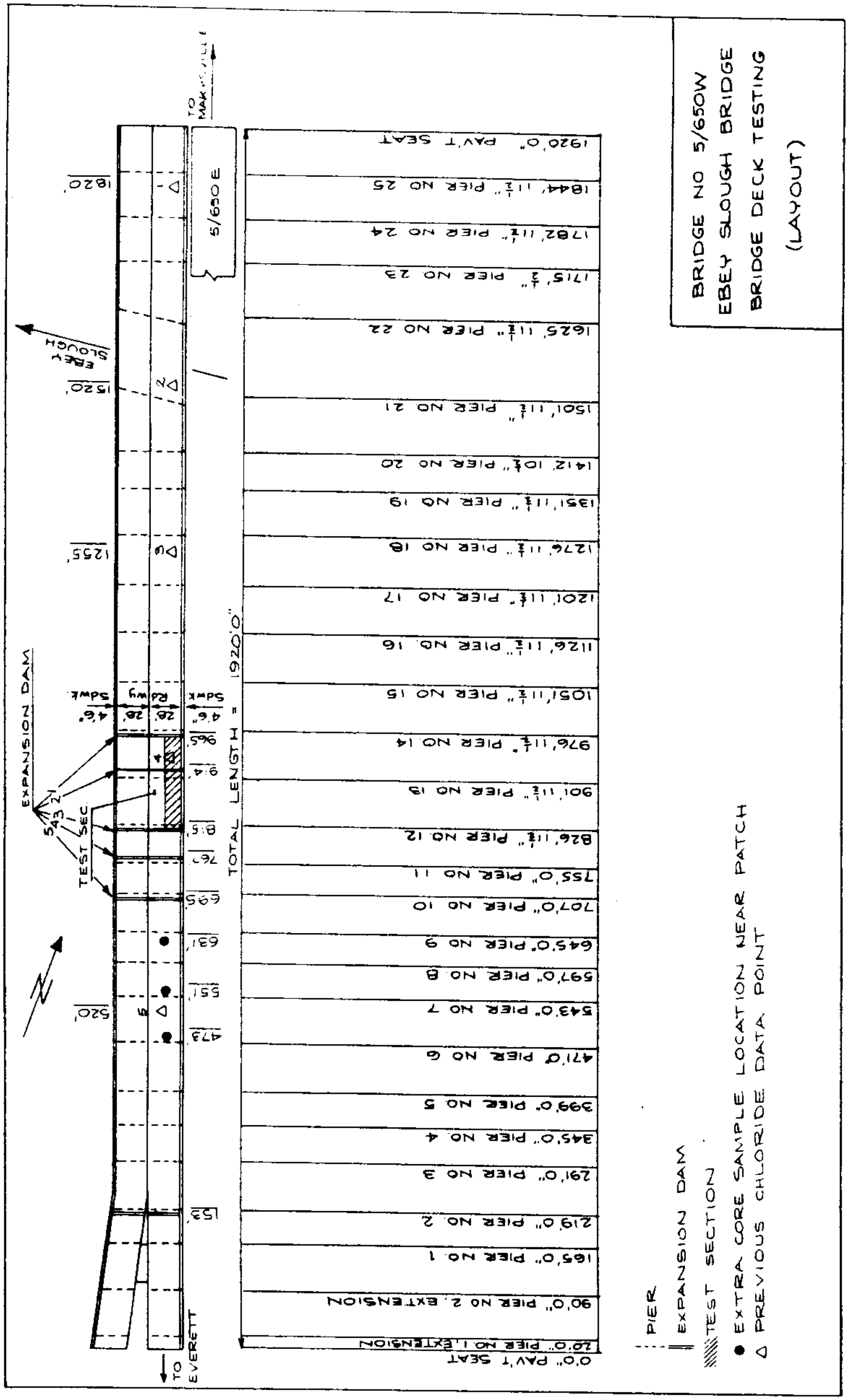

Figure 3. Location of Test Section and Extra Core Sampling at Ebey Slough Bridge Deck

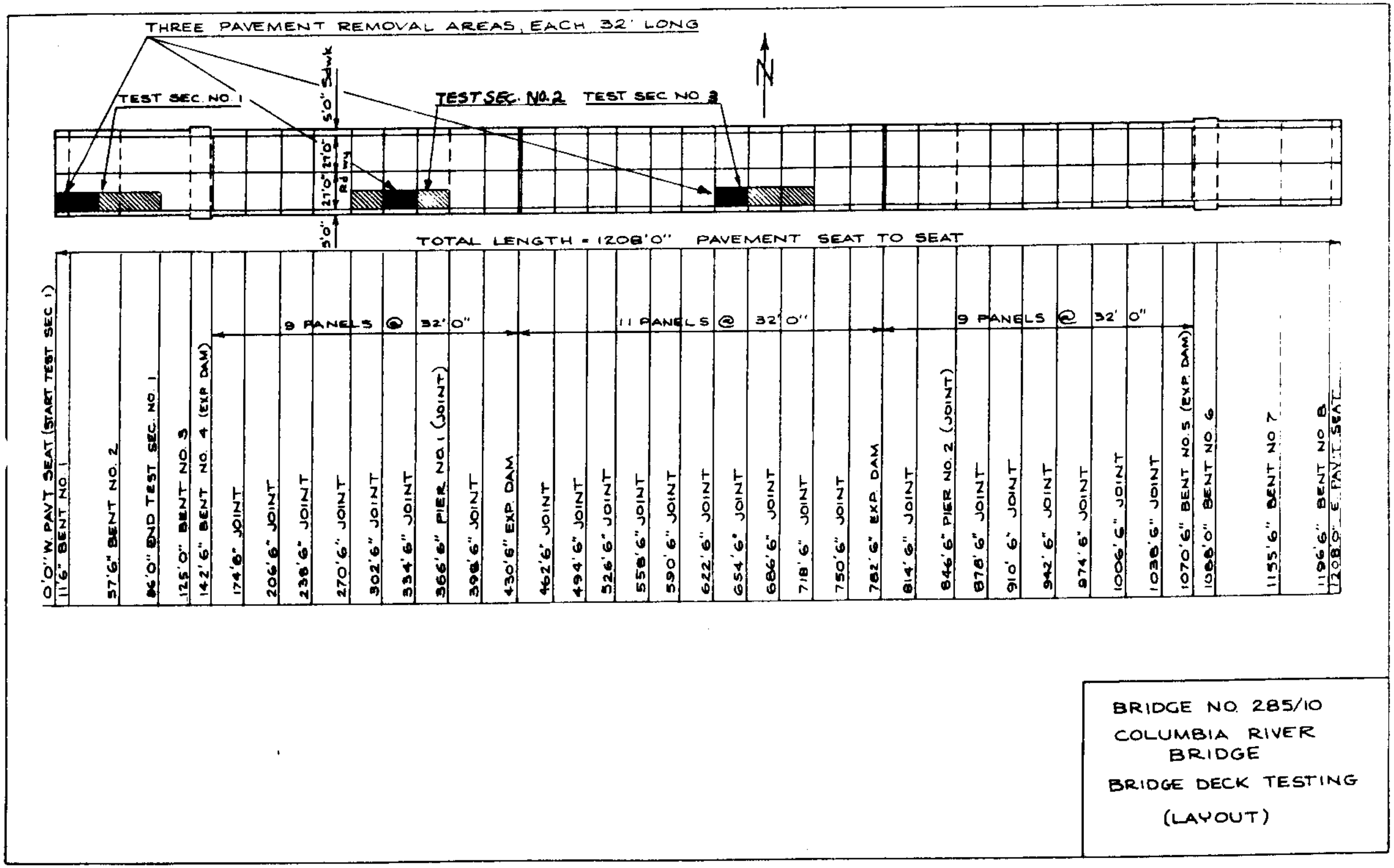

Figure 4. Layout of Revised Test Sections on Bridge 285/10 and Locations of Concrete Deck to be Exposed

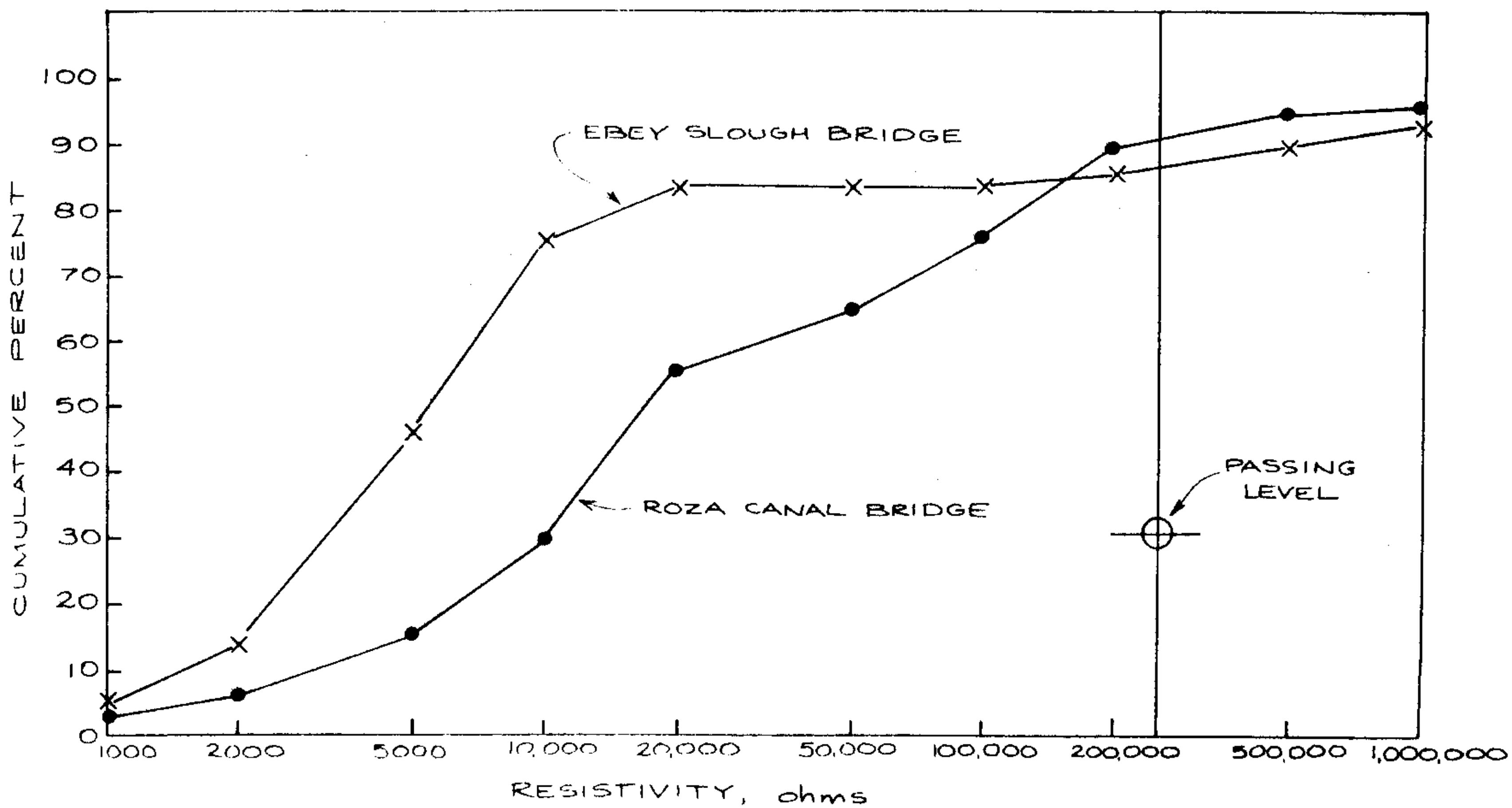

Figure 5. Electrical Resistivity Readings at Roza Canal and Ebey Slough Test Sites

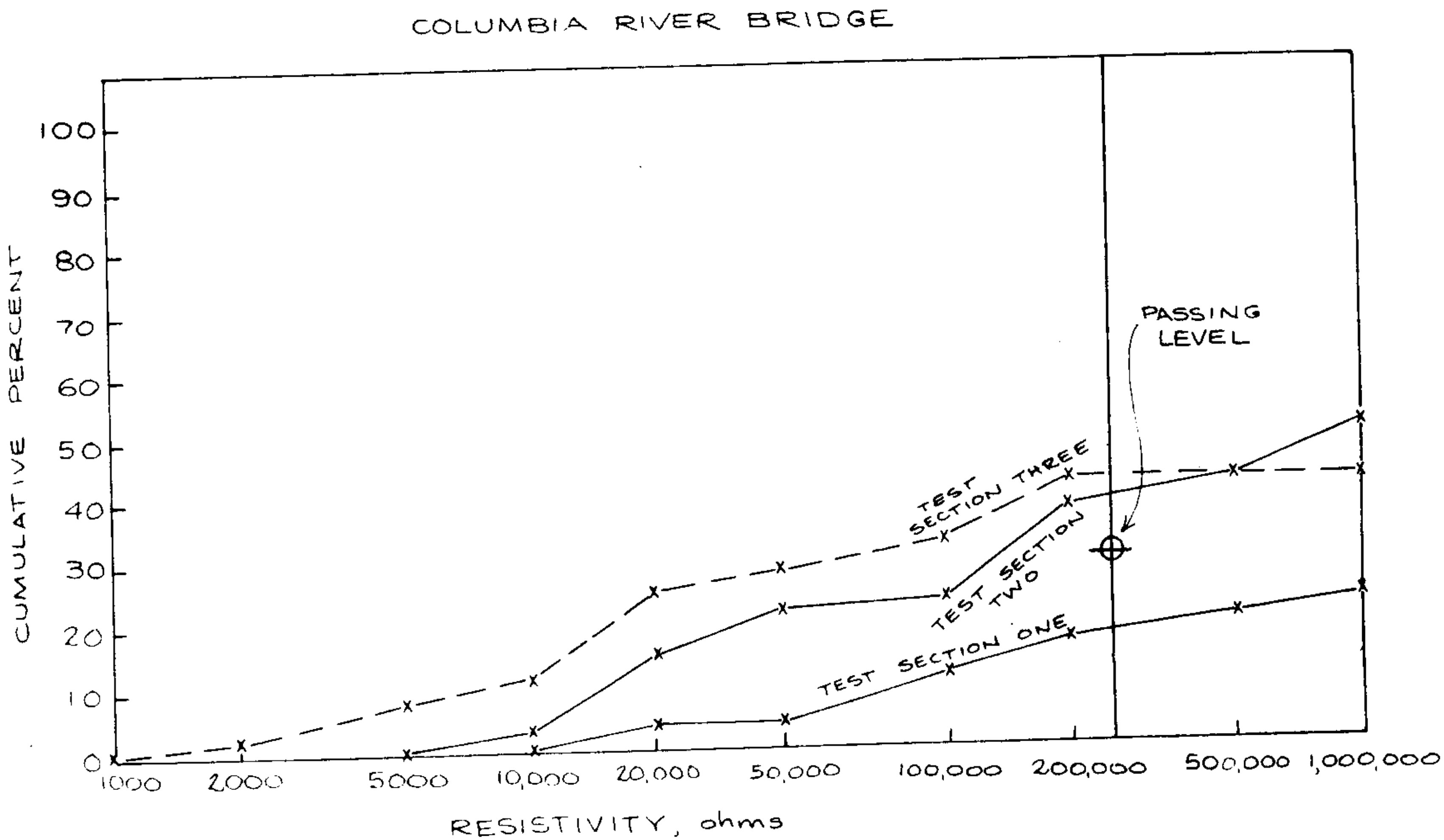

Figure 6. Electrical Resistivity Readings at Columbia River Bridge Deck

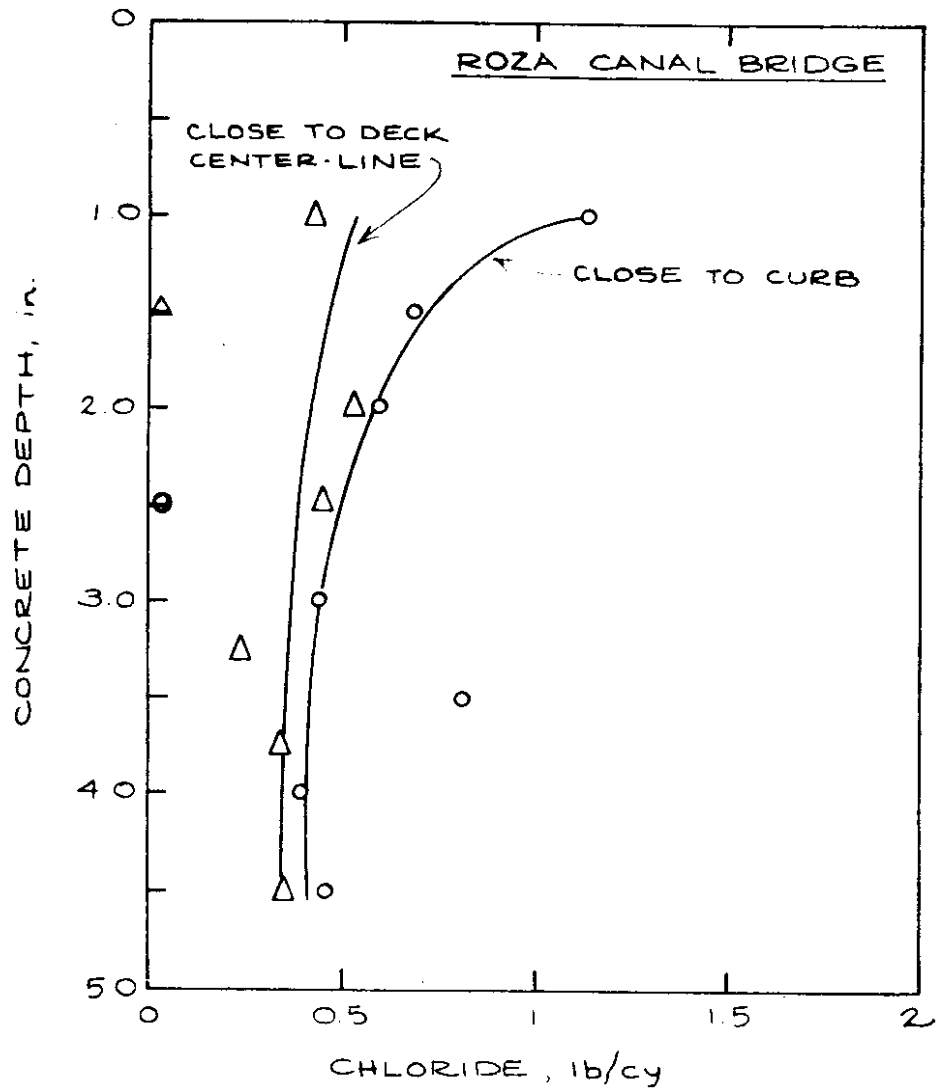

Figure 7. Chloride Content Versus Concrete Depth Profile at Roza Canal Test Site

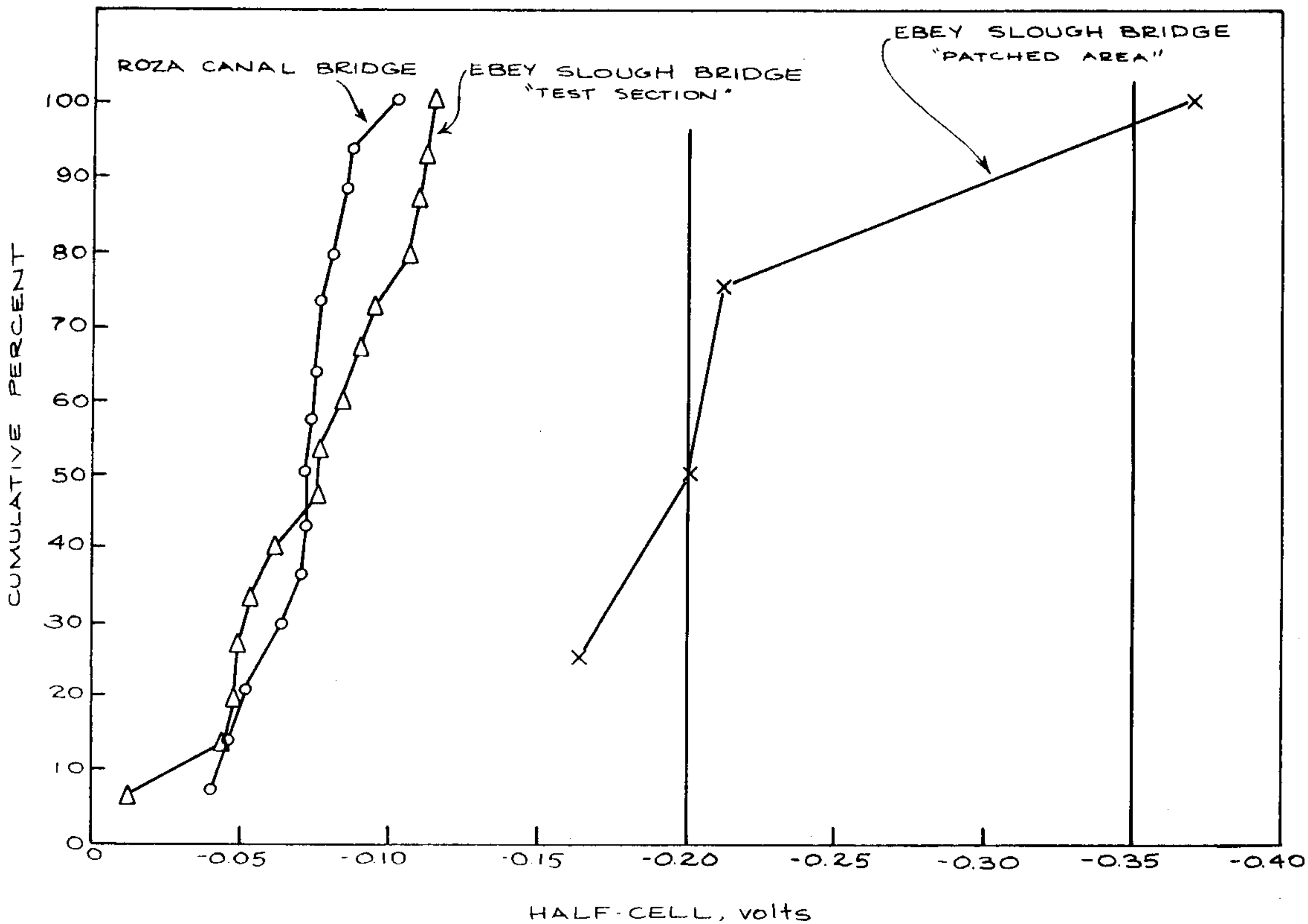

Figure 8. Electrical Potential Readings at Roza Canal and Ebey Slough Test Sites

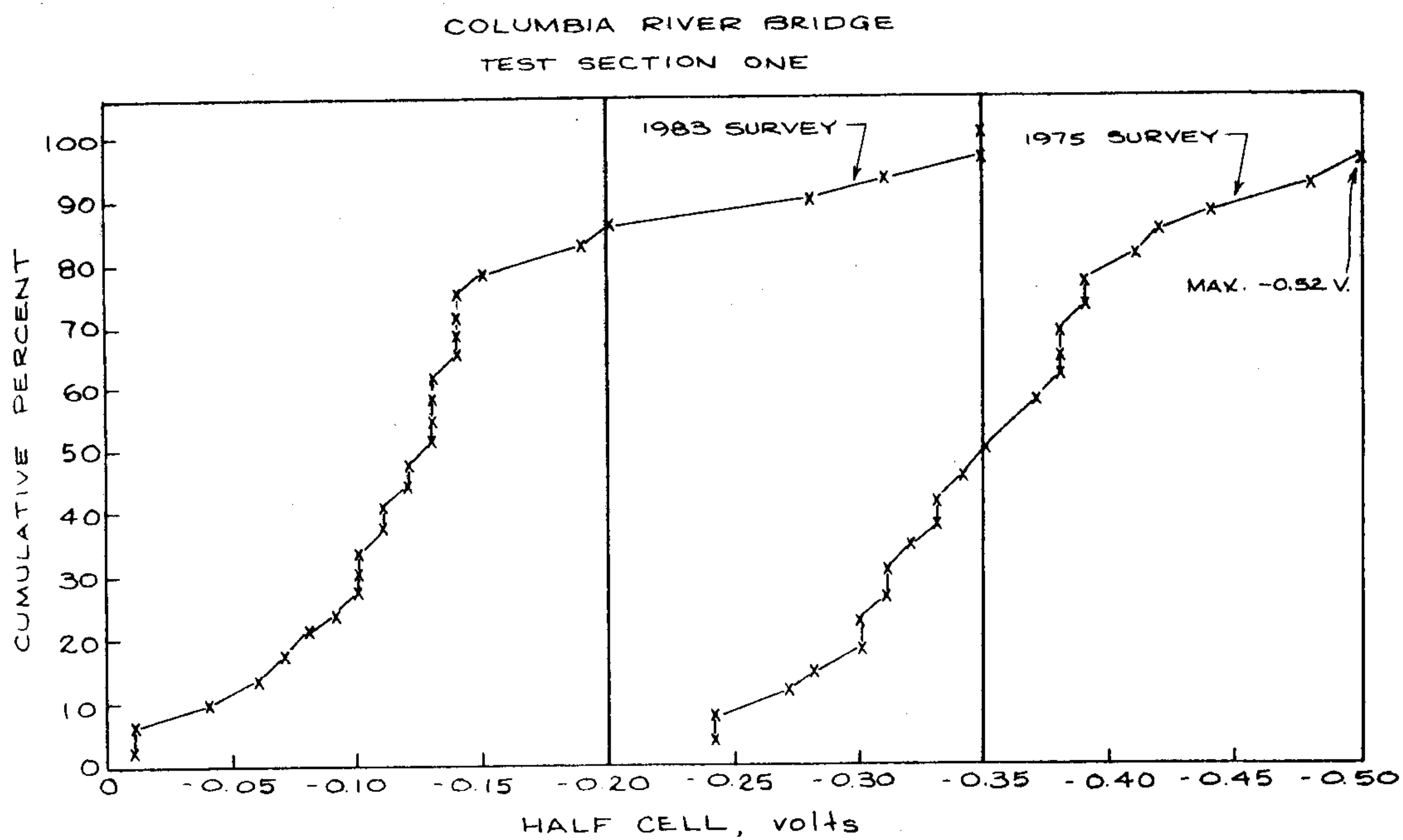

Figure 9. Half-Cell Readings for 1975 and 1983 (Columbia River Bridge Deck, test section 1)

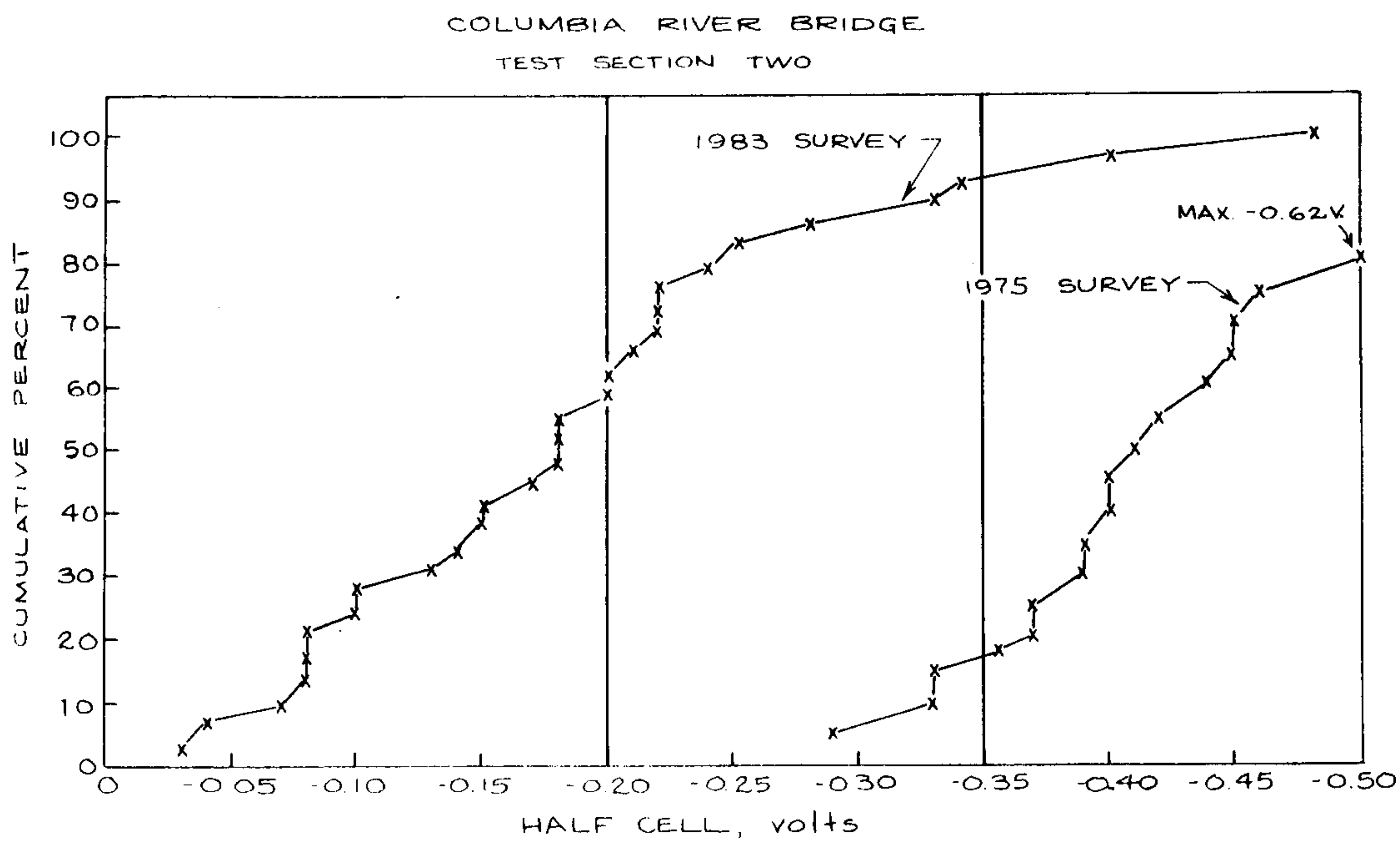

Figure 10. Half-Cell Readings for 1975 and 1983 (Columbia River Bridge Deck, test section 2)

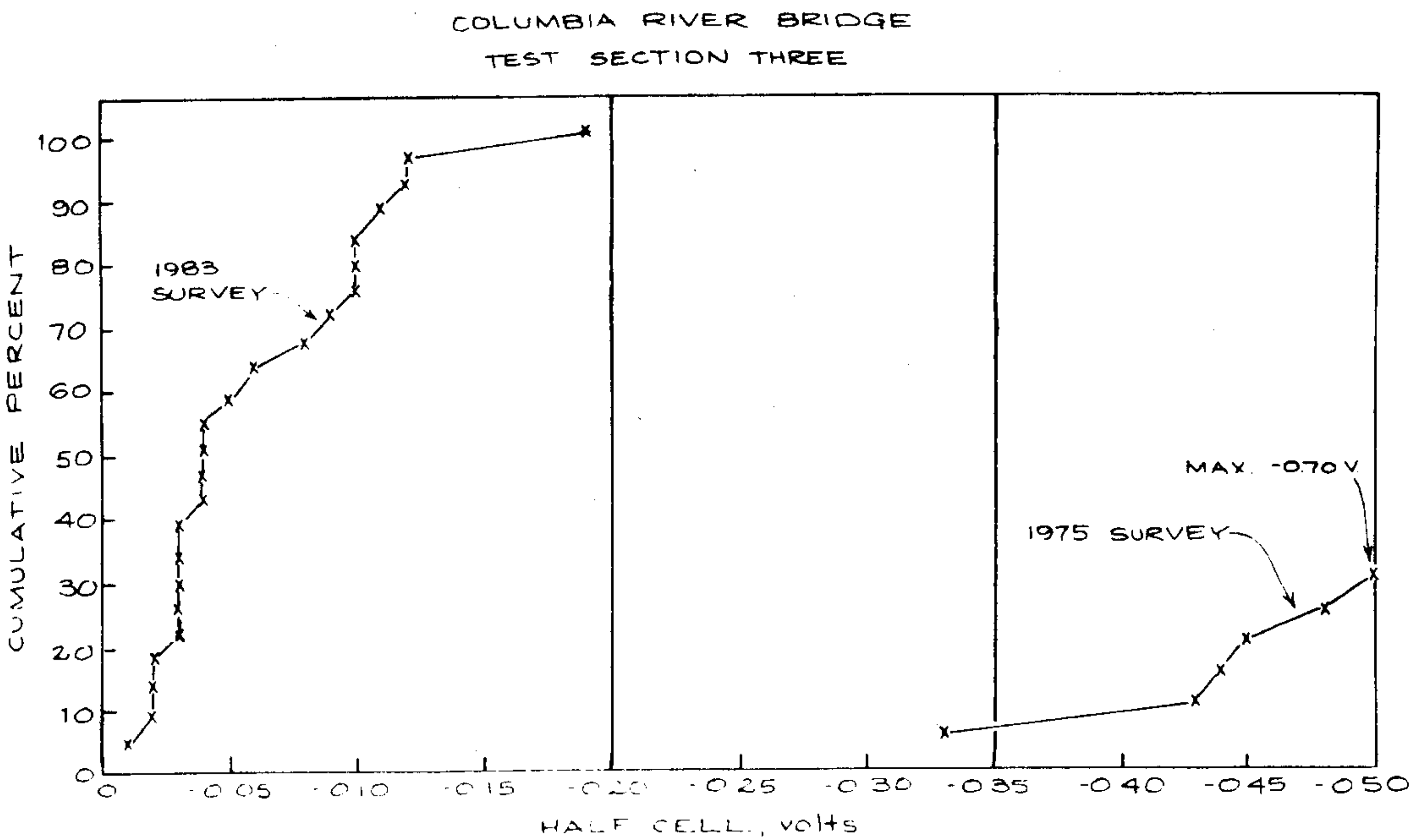

Figure 11. Half-Cell Readings for 1975 and 1983 (Columbia River Bridge Deck, test section 3)

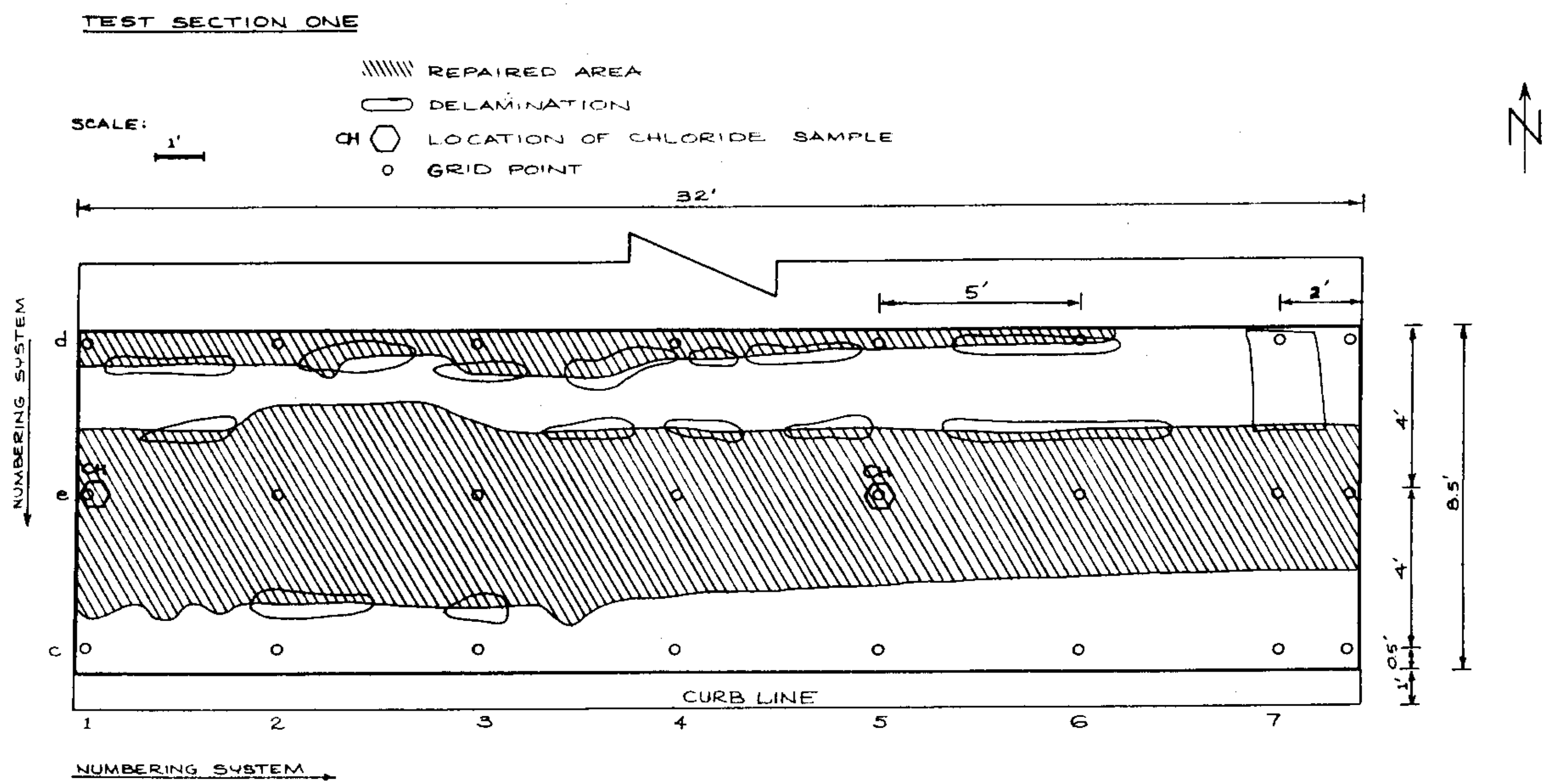

Figure 12. Repair Job and Defects on Exposed Concrete (Columbia River Bridge Deck, test section 1)

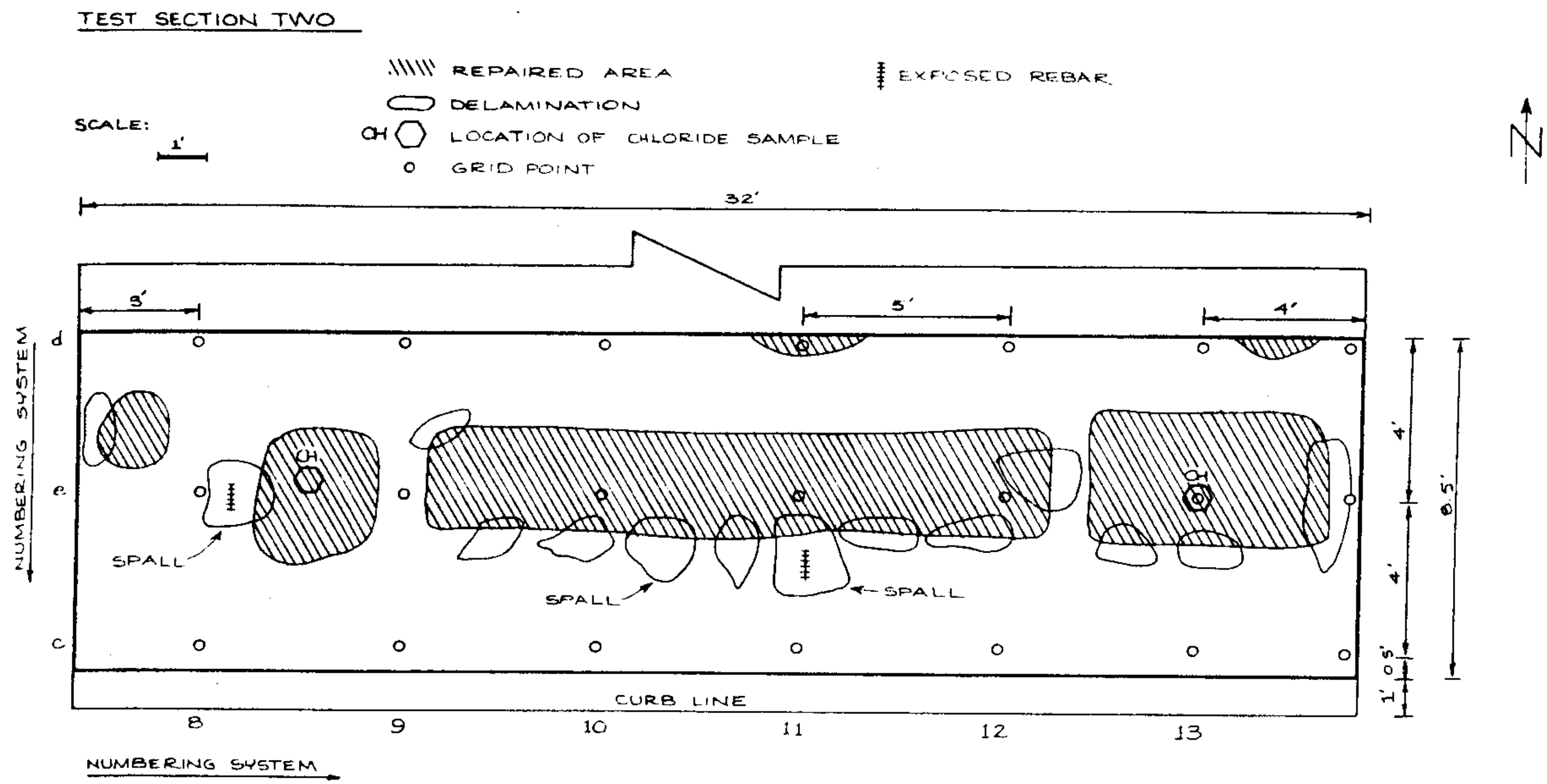

Figure 13. Repair Job and Defects on Exposed Concrete (Columbia River Bridge Deck, test section 2)

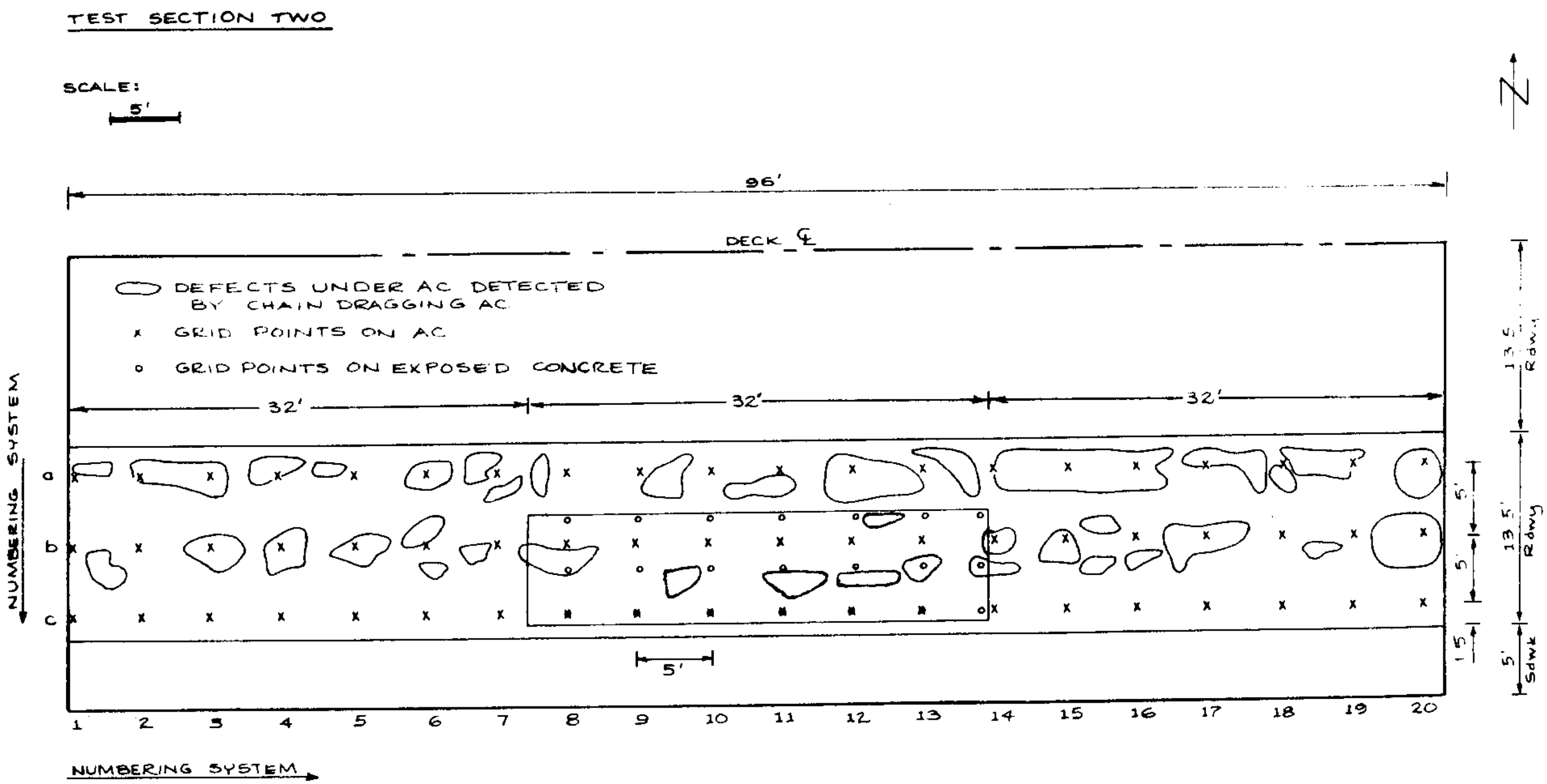

Figure 14. Defects Under Pavement Detected by Chain Dragging Asphalt Overlay (Columbia River Bridge Deck, test section 2)

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Table 1. Results of Chloride Content Measurement in 1974 and 1983 on Ebey Slough Bridge Deck

Sample No.	Sample Location		Cl^- Measured in 1974 (lb/cy)	Cl^- Measured in 1983 (lb/cy)[a]	Increase in Cl^-		Average Increase in Cl^-	
	Distance from North Pavement Seat (ft)	Distance from Curb (ft)			lb/cy	%	lb/cy	%
1	100	10.0	0.66	0.75	0.09	14	0.59	33
2	400	7.5	1.08	1.40	0.32	30		
3	665	8.5	1.28	0.14[b]	--	--		
4	990	16.0	2.31	>2.58[c]	>0.27	>12		
5	1400	12.5	2.24	<3.90[d]	<1.66	<74		

a: Resamples of 1974 survey, sampling depth in concrete 0.50 in to 1.25 in

b: Erratic result

c: Sample 23-a, measured at 0.95 in to 1.45 in

d: Sampling depth 0.50 in to 1.00 in

Table 2. Results of Chloride Content Measurement in 1975 and 1983, Columbia River Bridge Deck

Sample No.	Sample Location		Cl^- Measured in 1975 (lb/cy)	Cl^- Measured in 1983 (lb/cy)[a]	Increase in Cl^-		Average Increase in Cl^-	
	Distance from * (ft)	Distance from Curb (ft)			lb/cy	%	lb/cy	%
1	11	7	3.45	5.20	1.75	51	1.46	64
2	336	5	1.53	2.69	1.16	76		

*: Distance from east-end of the exposed concrete of test Section 3

a: Resamples of 1975 Survey, sampling depth in concrete, 1" to 1.5"

Table 3. Results of Chloride Content Measurement at the Exposed Concrete, Columbia River Bridge Deck

Test Section	Sample No.	Nature of Concrete	Depth in Concrete (in)	Cl^- (lb/cy)
1	1-e	Patch	½ - 1	0.62
			1 - 1½	1.05
	5-e	Patch	½ - 1	0.98
			1 - 1½	--
2	8,9-e	Patch	½ - 1	0.73
			1 - 1½	1.11
	13-e	Patch	½ - 1	0.42
			1 - 1½	1.14
3	2-d	Original	1 - 1½	0.86
			1½ - 2	0.31
			2 - 2½	0.26
	6,7-c	Original	1 - 1½	0.15
			1½ - 2	0.19
			2 - 2½	0.83

Table 4. Chloride Content Versus Depth at Roza Canal Test Site

Sample 2-d		Sample 11-a	
Concrete Depth (in)	Cl^- (lb/cy)	Concrete Depth (in)	Cl^- (lb/cy)
1/2 - 1	1.13	1/2 - 1	0.43
1 - 1 1/2	0.69	1 - 1 1/2	0.00
1 1/2 - 2	0.59	1 1/2 - 2	0.54
2 - 2 1/2	0.02	2 - 2 1/2	0.45
2 1/2 - 3	0.44	2 1/2 - 3 1/4	0.24
3 - 3 1/2	0.81	3 1/4 - 3 3/4	0.34
3 1/2 - 4	0.39	3 3/4 - 4 1/2	0.35
4 - 4 1/2	0.46	-	-

Table 5. Chloride Intrusion into Concrete of Deck Since Installation of Membrane

Bridge Name	Sampling Location on Deck	Concrete Depth Range (in)	No. of Samples	Ave. Cl^- Intrusion (lb/cy)	Membrane years of Service
Ebey Slough	Marysville side	0.5-1.25	1	0.23	9 1974-1983
	Everett side (around AC patches)		3	<1.66*	
Columbia River	Repaired concrete	0.5-1.0	4	0.39	7 1976-1983
Roza Canal	Traffic lane	0.5-1.0	1	0.24	14 1969-1983
	Shoulder		1	0.83	

*Sampling depth 0.5 in to 1.00 in.

Table 6. Range of Variables to be Evaluated in Future Studies

Site	1	2	3	4	5	6	7	8
Climate	high	low	M	M	M	M	M	M
Traffic Level	M	M	high	low	M	M	M	M
Overlay Thick-ness	M	M	M	M	high	low	M	M
Grade	M	M	M	M	M	M	high	low

PERFORMANCE EVALUATION OF WATERPROOFING MEMBRANE PROTECTIVE SYSTEMS FOR CONCRETE BRIDGE DECKS

VOLUME II

EXPERIMENT DESIGN AND TEST RESULTS

I. PART ONE

DATA ASSIMILATION AND EXPERIMENT DESIGN

LIST OF FIGURES

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1. INTRODUCTION

The purpose of this phase of the study entitled "Performance Evaluation of Waterproofing Membrane Protective Systems for Concrete Bridge Decks" is to design a field experiment and prepare a plan in order to give a reasonable preliminary assessment of the effectiveness of certain membrane systems applied on decks selected by WSDOT. It is also the aim of the investigation that the information to be collected during the testing period be used to provide guidelines for a continuing long-range evaluation of the waterproofing membrane installations.

WSDOT's recent specifications for protection of concrete bridge decks against chloride deicing chemicals recommends application of membrane and asphalt concrete (AC) overlay systems mainly for waterproofing of repaired bridge decks. Generally, WSDOT prefers the use of impermeable concrete overlays, such as dense and latex modified concrete overlays, rather than the membrane system. However, the use of the latter is sometimes required as an expedient method. An example is when traffic volumes influence the choice of bridge deck protective systems, or where a bridge widening or reconstruction involves an existing asphalt overlay. A requirement for application of waterproofing membrane and AC overlay is the monitoring of installations, since few data are yet available on the service performance and effects of the system on the continuing corrosion of reinforcing steel.

From among the many bridge decks in Washington that have already received a waterproofing membrane and asphalt concrete treatment, three bridges were selected by the WSDOT Bridge Branch for evaluation. The decks selected depended upon previous test data available on each as well as geographical location, age of original structure, and time since the overlay was applied. The following bridges were selected:

1. Bridge 285/10 (Columbia River)
2. Bridge 82/106 (Roza Canal)
3. Bridge 5/650 W (Ebey Slough)

After available data on the candidate bridges were assembled and examined, a literature survey was conducted to obtain information developed from previous investigations relating specifically to the methods of evaluating the effectiveness of waterproofing membrane systems and which are relative to this study. A summary of previous investigations is included in Appendix A. Included in Appendix A is also the result of a computer literature search conducted at the WSDOT technical library regarding methods of membrane waterproofing reinforced concrete decks. Following this step, test sections on each bridge deck were located and a plan was developed to test the waterproofed decks in conjunction with the WSDOT Materials Laboratory.

Generally, the goal of the investigation is twofold -- first, to determine if the membrane and AC overlay systems on the selected decks have totally waterproofed the concrete decks against the ingress of chloride and water, and second, to determine if the systems are effective in decreasing the intensity of continuing corrosion of the reinforcing steel. The latter corresponds to waterproofing existing decks, which does not require removing chloride-contaminated concrete.

In an attempt to determine these unknowns, several field evaluation methods are proposed in the testing plan while taking the capabilities of the WSDOT Materials Laboratory, as well as actual field conditions, into consideration. Stated briefly, the following is a listing of the proposed field evaluating methods:

a. Electrical resistivity
b. Chain dragging
c. Electrical potential
d. Core sampling
e. Rebar depth measurement (pachometer survey)
f. Chloride content measurement

Electrical resistivity testing determines the degree of impermeability of a membrane. This can also be achieved by chloride content measurement, if the baseline level of chloride is already available. Chain dragging defines the areas of delamination in the concrete. Here again, the baseline level of delamination is required for a meaningful evaluation of membranes. Electrical potential measurement indicates the general level of corrosion activity of the reinforcing steel. A comparison with "before" condition data is not always necessary but could be very useful in determining the effectiveness of membranes. It should be mentioned that the electrical potential measurement cannot be used to determine the rate of corrosion, which could be the best indication of the state of continuing rebar corrosion. Corrosion rate measurement is not included in this study for two reasons. First, a longer period of time is needed for monitoring the rate of corrosion. Second, although very promising , data are not yet available on the quantitative accuracy of the measurement. Measurement of rebar depth is needed prior to sampling concrete for chloride analysis. In addition, it could determine if the deteioration of concrete and/or corrosion of the reinforcing steel are associated with thin concrete covers.

In order to inspect the test sections on the selected bridges in advance of field testing, field trips to the sites were made. This was accomplished after a tentative test plan was submitted to WSDOT and the plan was generally approved by both the Bridge Branch and the Materials Laboratory. Reports of these field trips are included in Appendix G.

2. SUMMARY OF WSDOT SPECIFICATION FOR MEMBRANE WATERPROOFING CONCRETE BRIDGE DECKS

A full description of WSDOT specifications for membrane waterproofing concrete bridge decks is included in Appendix B. Generally, one of the following membrane systems will be selected by the contractor:

System A: A factory laminated sheet composed of either suitably plasticized coal tar or rubberized asphalt reinforced with a polypropylene fabric and primed in accordance with the manufacturer's recommendation.

System B: A hot-applied rubberized elastomeric membrane with primer if required by the manufacturer.

System C: A hot-applied reclaimed rubber asphalt membrane.

Prior to application of the primer or liquid membrane, the entire deck shall be free of all foreign material such as dirt and grease. All dust and loose material shall also be removed from the deck with compressed air. Surface defects such as spalled areas, cracks, protrusions which decrease membrane effectiveness by puncturing, and stretching shall be corrected before application of the membrane. The deck shall be surface-dry at the time of the application of the membrane and ambient air temperature shall be above 50 degrees F.

All membrane systems shall be protected from damage due to the paving operation. The methods of membrane protection for systems A and B shall be recommended by the manufacturer. The method of protection for System C will be a polypropylene material fabric which will cover the membrane prior to overlaying with asphalt concrete. The manufacturer will also specify the procedures for placing asphalt concrete overlay, such as type of paving machine and laydown temperature of asphalt concrete in order to protect the membrane.

In the engineers' judgment, completed sections of waterproofing membrane should be evaluated for waterproofing effectiveness prior to the application of membrane protection. Electrical resistance testing should be used for this purpose (WSDOT Test Method No. 413A). Any portion of the membrane having resistance readings of less than 100,000 ohms shall be repaired. In addition, those membranes which provide less than 70% readings above 250,000 ohms shall be replaced or repairs may be made to bring the membrane to the acceptance level. After completion of the asphalt overlay, a final evaluation shall be made using electrical resistance testing having an acceptance level of 70% readings above 250,000 ohms and no single reading below 100,000 ohms .

3. BRIDGE SELECTION AND DATA ASSIMILATION

From among the many bridge decks in the State of Washington that have alrady received a waterproofing membrane and AC overlay, three bridges were selected by the WSDOT Bridge Branch to investigate the protective effectiveness of the membrane systems applied on them. The selected decks depended upon the previous test data available on each as well as geographical location, age of structure, and time since the overlay was applied. The following is a list of the test bridges:

1. Bridge 285/10 (Columbia River) near Wenatchee
2. Bridge 82/106 (Roza Canal) near Yakima
3. Bridge 5/650W (Ebey Slough) near Everett

Geographical location of the bridges is shown in Figure 1. The information provided by WSDOT on the condition of the three test bridges may be summarized as shown in Table 1. The table is self-explanatory and be summarized as shown in Table 1. The table is self-explanatory and contains general information on the deck concrete, membrane, AC overlay, types of tests and repairs previously conducted on the decks, etc. More information regarding the engineering properties of the test bridges, as well as the routine bridge inspection reports, can be found in Appendix C, which includes WSDOT's bridge information cards for the three test bridges.

3.1. Bridge 285/10 (Columbia River)

Bridge 285/10 was built in 1950. It is 1208 ft (366 m) long, carries State Route 285 and is located 0.3 miles (0.5 km) east of Junction State Route 28. Two types of spans, steel arch and concrete T-beam, are used in the superstructure. The plan and elevation of the bridge are illustrated in Figure 2. The deck slab in steel span is 6.5 in (16.5 cm) thick. The top mat consists of transverse no. 5 reinforcing bars at 6 in (15.2 cm) centers over longitudinal no. 4 bars at 30 in (76.2 cm) centers. The concrete deck was built in accordance with the old specification (i.e., small rebar cover thickness and high water/cement ratio), which did not provide waterproofing to the top mat reinforcing steel. In 1975, a detailed field condition survey, including visual survey, chloride content measurement, corrosion and delamination detection, was conducted on the deck [1]. The results of this survey revealed the chloride contamination and delamination of the deck concrete as well as the corrosion of the top mat reinforcing steel. A summary of the results of this investigation is shown in Table 2. Complete data from this survey are included in Appendix D. Due to the extensive deterioration, the deck was scheduled for repairs and subsequent waterproofing. In 1976, the delaminations were repaired by removing the unsound concrete and placing a grout on the excavated areas [2]. The location of the repaired areas on the deck are shown in Appendix E. After completion of the repair job, the deck was waterproofed with a hot-applied rubberized asphalt membrane and 1.8 in (4.6 cm) AC overlay.

The waterproofed deck has been inspected by the Bridge Branch annually and its condition has been reported as shown in Table 3.

3.2. Bridge 82/106 (Roza Canal)

Bridge 82/106 was built in 1969. It is 105 ft (31.8 m) long, carries State Route 82 and is located 6.3 miles (10.5 km) east of the Kittitas County line. The structure consists of two parallel bridges, L/N and L/S bridges, on westbound and eastbound highways. Each bridge is made of one single span supported by pretensioned concrete beams (see Figure 3 for plan & elevation). The deck slab is 6.5 in (16.5 cm) thick. The top mat consists of transverse no. 5 reinforcing bars at 6 in (15.2 cm) centers over longitudinal no. 4 bars at 11 in (27.9 cm) centers. The concrete deck was waterproofed with a waterproofing membrane (Petromat) and 1.5 in (3.8 cm) AC overlay before opening to traffic. No detailed field condition survey has been conducted on the deck, but the deck has been inspected routinely by the Bridge Branch annually and its condition has been reported as shown in Tables 4 and 5 for bridges L/S and L/N, respectively.

3.3. Bridge 5/650 W (Ebey Slough)

The bridge was built in 1954. It is 1920 ft (581.8 m) long, carries State Route 5 and is located 4.6 miles (7.7 km) north of Junction State Route

2. Two types of spans, concrete T-beam and steel girder, are used in the superstructure. The plan and elevation of the three segments of the bridge, constructed under different contracts, are shown in Figures 4, 5 and 6. The deck slab in the concrete span is 6.5 in (16.5 cm) thick. The top mat consists of transverse no. 5 reinforcing bars at 6 in (15.2 cm) centers over longitudinal no. 4 bars. Like Bridge 285/10, no measures were taken at the time of construction to waterproof the concrete deck. In 1973, the deck was waterproofed with a fabric waterproofing membrane (Petromat) membrane and 1.8 in (4.6 cm) of asphalt overlay. No major repair was done on the deck before waterproofing and no information is available to show the extent of contamination and/or deterioration of the deck concrete prior to the application of the membrane. In 1974, a visual survey and chloride analysis were conducted on the waterproofed deck. A summary of this survey is included in Table 6. Complete information on the chloride content, as well as the location of the concrete samples for chloride analysis, is provided in Appendix D.

The waterproofed deck has been given routine annual inspections by the Bridge Branch and its condition has been reported as shown in Table 7.

4. TESTING PLAN

It was decided that among the three bridges selected only Bridge 285/10 (Columbia River) will have a complete set of tests requiring overlay and membrane removal and replacement. The other two bridges, 82/106 (Roza Canal) and 5/650 W (Ebey Slough), will be inspected and tested, but without overlay and membrane removal. The reason behind this decision is to expedite the testing process on the latter bridges so the testing could be accomplished within one day with a minimum of disruption to traffic. Since Bridge 285/10 had already been tested in detail before the application of waterproofing membrane and had a considerable amount of "before" condition data, it was decided to remove the membrane and AC to collect similar, comparable data of "after" conditions as well.

4.1. Bridge 285/10

Three representative test sections, each 96 ft (29.1 m) long by 11.5 ft (3.5 m) wide, were located on the outside lane southbound, with a total area of AC and membrane removal of 3312 sq ft (304.1 sq m) (see Figure 7 for layout of the test sections). As shown in Appendix D, a portion of the deck (i.e., outside lane southbound beginning with the west pavement seat and ending with the fourth expansion dam on the deck counted in the direction of surveying) was surveyed for delaminations, electrical potential of rebar, and chloride content before repairing the delaminations and waterproofing the deck. Taking this into consideration, the test sections were located on those areas of the deck which had already been tested. However, due to a small number of chloride samples previously taken, only one chloride content data point was available for each test section. Another factor influencing the location of the test sections was the roughness of concrete surface upon which the membrane was placed. Studies conducted by the New York State Department of Transportation [3] indicate that serious consideration should be given to a requirement for surface smoothness before any membrane is placed on a rehabilitated bridge deck. It was noticed that the concrete of test section 1 was heavily repaired and patched prior to installation of the membrane, whereas the concrete of test section 2 and test secction 3 had consideration was given to convenience for testing and replacement. In determining the length of the test sections, the estimated testing budget was taken into consideration.

4.1.1. Sequence of Testing Activities

The testing activities to be conducted by the WSDOT Materials Laboratory on each test section on bridge 285/10 and their sequence will be as follows (see also Section 5 for general testing practice):

(A) Photograph and map the AC overlay surface condition, including cracks of all types, raveling, patching, shoving, etc. Photograph and record the condition of the curbs, such as cracks and spalling. Photograph and record the condition of the deck from below, such as dry cracks, seepage, efflorescence, spalling, etc.

(B) Chain drag the AC overlay to locate possible debonding (or blisters) between the concrete slab and AC overlay. Map and outline the areas of debonding. (It should be mentioned at this point that the dull sound resulting from striking the AC surface could be related to some of the delaminations or scales of the concrete deck. A second chain dragging on the bare concrete surface, as explained in item (G) below, will clarify the source of the dull sound.)

(C) Conduct electrical resistivity testing on AC overlay using the grid pattern shown in Figure 8 and record the resistivity of the membrane at the intersecting points of the grid. This pattern is adapted from ASTM D3633, which requires 5 ft (1.5 m) spacing between data points and at least 1.5 ft (0.5 m) clearance from curbing.

(D) Take four core samples through AC and membrane, two core samples from the areas of high resistivity and two core samples from the areas of low resistivity readings (i.e., > 500,000 and < 100,000 ohms). Use the dry method for core sampling. Examine the core samples, and characterize and record the condition of membrane using the data form shown in Figure 9. In addition, obtain 4 concrete samples from the level of top mat reinforcing steel for chloride analysis from either the core locations or areas in close vicinity to the core locations.

(E) Remove the existing AC overlay and membrane (if possible remove 1 sq ft or 0.1 sq m of the AC overlay by jack-hammering in areas of high and/or low resistivity readings with the membrane intact to directly evaluate the condition of the membrane). Sandblast concrete surface to clean the surface sufficiently to expose cracks and other distress of the concrete.

(F) Photograph and map the exposed concrete surface condition, including all cracks, spalling, patching, scaling, popouts, exposed rebar, and any other defects noted.

(G) Chain drag the entire exposed concrete surface, noting all apparent delaminations. Map and outline the areas of delaminations.

(H) Conduct a pachometer survey on the concrete and record the average depth of rebar at five points on the delaminated surface and five points on the sound concrete. If possible, use the intersecting points of the grid shown in Figure 8. It should be mentioned that the results of research conducted by the New York State Department of Transportation [4] indicate that the presence or absence of deterioration of concrete decks is related to cover depth. It would be desirable to take readings of rebar depth on the grid pattern over the entire exposed section. However, since the intent is not to study the effect of concrete cover, the number of readings should depend on the time and staff available for the study.

(I) Conduct electrical potential testing on the entire exposed section using the grid pattern of the electrical resistivity testing as shown in Figure 8 and record the temperature. The intent is to collect comparable half-cell data to the electrical resistivity data (if possible). Extra measurements between the grid points should be taken in the areas of high corrosion activity where adjacent readings exhibit differences exceeding 0.150 v.

(J) Obtain four core samples of concrete, two from the area of high half-cell readings and two from the area of low half-cell readings (i.e., > 0.35 and < 0.20 volts). The samples should be taken directly over the top mat reinforcing steel, avoiding drilling through the rebar. Examine the core samples and characterize and record the appearance of the concrete as well as the corrosion state of the steel beneath the concrete. In addition, determine the chloride content at the level of rebar using either the concrete of the core samples or concrete in close vicinity to the core locations.

(K) A chloride content versus depth profile should be taken in the vicinity of the previous chloride data point taken in 1975 (use WSDOT procedures for this as explained in [12].

(L) Install monitoring devices (if any) for future long-range evaluation on the membranes. This could include rate of corrosion measurement probes installed at the level of reinforcing steel and the areas of high half-cell readings (see [5] for more information). If possible, a permanent connection should be established to the rebar to facilitate future measurements.

(M) Apply a new membrane on the exposed area on the test section. This could be an opportunity to apply three different experimental membranes on the three test sections and start long-range monitoring of their behavior under service life.

(N) Upon completion of membrane application, but prior to repaving AC overlay, take electrical resistivity readings as described in item (C) above.

(O) Photograph and record any damage done to the membrane during paving the test section with asphaltic concrete overlay.

(P) After completion of the overlay, chain dragging should be repeated once again as explained in item (B) above to check the integrity of the new membrane and AC patch for possible debonding.

(Q) Following the chain dragging, take electrical resistivity readings, once again, to determine if any damage to the membrane has occurred during construction of the overlay.

(R) Electrical resistivity readings could be taken at intervals of time after the original installation to continuously monitor the service performance of the membranes placed on the test sections.

4.2. Bridges 82/106 and 5/650 W

As mentioned earlier, no overlay and membrane removal was planned for these two bridges. On Bridge 82/106, a test section 105 ft (31.8 m) long by 18 ft (5.5 m) wide was located on almost the entire outside lane and shoulder of the L/S bridge, due to the relatively small length of the bridge (i.e., 105 ft or 31.8 m total length) as shown in Figure 10. As explained in Section 3.2, no data on the condition of the deck concrete and membrane were available for Bridge 82/106. However, the membrane and AC overlay were placed on the deck prior to opening the deck to traffic. Thus, any sign of chloride contamination of the deck concrete and/or corrosion of the rebar would be an indication of unsatisfactory behavior of the waterproofing membrane at the time of testing, eliminating the need for chain dragging the bare concrete.

On Bridge 5/560 W, only one test section, 150 ft (45.5 m) long and 13 ft (3.9 m) wide, was located on the inside lane southbound as illustrated in Figure 11. The entire "before" condition data on this bridge are limited to five chloride content measurements taken almost every 300 ft (90.9 m) in the direction parallel to the axis of the deck on the inside lane southbound after waterproofing the deck (for more information see Appendix D). Thus, only one chloride content data point, which was also the highest among those measured, could be identified on the test section. It is believed that this section of the deck is the most appropriate place to evaluate the condition of the membrane, since the area adjacent to that and toward Everett, which is on grade, is highly patched and the membrane may be distorted, and the area in the other direction (i.e., toward Marysville) has much lower chloride concentration. Since no data were available regarding the delamination of concrete prior to waterproofing the deck, chain dragging the bare concrete to detect delaminations would not be very meaningful in this case. But measurement of the electrical potential of the rebar could at least indicate if corrosion is continuing after waterproofing and at the time of testing, regardless of the availability of baseline data.

4.2.1. Sequence of Testing Activities

The testing activities to be conducted by the WSDOT Materials Laboratory on each test section on Bridges 82/106 and 5/650 W and their sequence will be as follows (see also Section 5 for general testing practice):

(A) Photograph and map the AC overlay surface condition, including cracks of all types, raveling, patching, shoving, etc. Photograph and record the condition of the curbs, such as cracks and spalling. Photograph and record the condition of the deck from below, such as dry cracks, seepage, efflorescence, spalling, etc.

(B) Conduct electrical resistivity testing on AC overlay and over the entire test section using the grid patterns shown in Figures 12 and 13 for Bridges 82/106 L/S and 5/560 W, respectively. The grid patterns are adopted from ASTM D3633, which requires 5 ft (1.5 m) spacing between data points and a minimum of 1.5 ft (0.5 m) clearance from curbing.

(C) Obtain a minimum of 15 core samples through AC and membrane at the intersecting points of the grid used for electrical resistivity testing (use dry coring). The location of the core samples should be randomly selected and spaced out over the entire width and length of the test section. Examine the cores, and characterize and record the condition of the membrane using the data form shown in Figure 9.

(D) Conduct electrical potential testing in the core locations where the membrane has been punctured or removed and record the temperature. The number of the readings will be equal to the number of the holes drilled.

(E) Obtain a minimum of 15 concrete samples from the level of top reinforcing steel for chloride analysis. If possible, use the core locations for this purpose. If not, use areas in close vicinity to the core locations.

(F) On Bridge 5/650 W, determine the chloride content in the vicinity of each previous chloride data point taken in 1974 on the entire deck and at the same depth for comparison. On Bridge 82/106, chloride content versus depth profile as deep as 5 in in the concrete should be taken on two locations selected randomly on the deck. This is to determine the original chloride content of the deck concrete caused by the aggregate only (if any).

(G) Waterproof core locations with a selected material and patch.

4.3. Discussion

The preceding test plans, explained in Sections 4.1.1 and 4.1.2, involve many activities; therefore, planning and scheduling of each testing site should be carefully done to minimize the disruption of traffic. On Bridge 285/10, which requires removal and replacement of existing AC overlay and membrane, those activities prior to pavment removal as well as pavement removal and cleaning (i.e., items (A) through (E) in Section 4.1.1) could possibly be accomplished in one day. The second day could be dedicated to activities performed after pavement removal and before membrane installation (i.e., items (F) through (L) in Section 4.1.1. Finally, on the third day, the installation of membrane and AC waterproofing system could be accomplished, as well as testing required during and after installation (i.e., items (M) through (R) in Section 4.1.1).

On Bridges 82/106 L/S and 5/650 W, which do not require removal of AC overlay and membrane system, the whole testing program can possibly be accomplished in one day with a minimum of disruption to traffic.

5. TYPES OF EXPERIMENTATION AND GENERAL TESTING PRACTICE

Section 4 dealt with different types of experimentation to be conducted by the WSDOT Materials Laboratory and their sequence within the testing plan. In this section, the emphasis will be on general practice of especially those tests not normally included in WSDOT's routine inspection of bridge decks but which are required as part of the testing plan.

Stated briefly, the different experiments to be conducted in this study could be categorized as follows:

A. Experimentations conducted routinely by WSDOT for inspection of bridge decks.

A-1. Visual inspection of AC and/or PCC to detect surface distress.

A-2. Chain dragging AC and/or PCC to detect debonding and delaminations, respectively.

A-3. Chloride analysis of concrete samples obtained from bridge deck to determine the content of chloride at rebar level and/or to obtain chloride content versus depth profile.

A-4. Electrical resistivity testing of membrane to evaluate the permeability of the membrane under service. (This test is used by the WSDOT Materials Laboratory to evaluate waterproofing effectiveness of newly installed membrane/pavement systems.)

B. Experiments not conducted routinely by WSDOT for inspection of bridge decks.

B-1. Core sampling AC and membrane and/or deck concrete to characterize the condition of membrane and/or concrete above the rebar, respectively.

B-2. Electrical potential testing to detect corrosion of top mat reinforcing steel.

B-3. Pachometer survey to determine the depth and/or location of rebar in the concrete.

5.1. Electrical Resistivity Testing

A full description of electrical resistivity testing has been published by ASTM (see Appendix F for ASTM D3633). This test is also adopted by the WSDOT Materials Laboratory and is listed as Test Method No. 413A. The basic configuration of the electrical circuit for electrical resistivity testing is illustrated in Figure 14.

Several key points of the testing procedure as reported in ASTM [6] include at least the following:

(A) The surface to be tested should be free of all foreign material by sweeping or scraping, or both. Water should not be used for cleaning and the surface must be dry prior to testing.

(B) Excessive moisture in the pavement and on top of the membrane can cause invalid readings. The resistance between any two sites may be checked before actual testing to assure a dry condition. To accomplish this, the ohmmeter should be attached to two probes, rather than one, and the reinforcing steel. Immediate low readings (10,000 $\pm$ ohms) will be an indication of excessive moisture in the pavement and on top of the membrane.

(C) A direct connection from the ohmmeter to the top mat of reinforcing steel is desirable.*

(D) To check for a completely satisfactory circuit and especially to ensure a good contact with the reinforcing steel, the probe should be placed on exposed concrete deck curbing. In this case, the resistance reading on the ohmmeter will normally range from 1,000 to 3,000 ohms. The probe should then be placed at several locations along the curb and the resistance reading must remain relatively constant.

(E) Water used to saturate the sponge or wet the concrete surface at test locations should be mixed with 1 oz/gal (8 ml/l) of wetting agent to break the surface tension and promote the penetration of the water through the bituminous pavement.

Some of the problems that can occur in measuring resistivity of the membrane which could cause invalid readings are illustrated in Figure 15 [7]. As shown in the figure, if the AC overlay is moist and water is standing on top of the membrane, a short circuit to the reinforcing steel can occur through a deck drain, a steel expansion device, or around the edge of the membrane adjacent to the curb, which causes low resistivity readings.

It should be kept in mind that the intent of the test is to identify both the defective and sound areas in the membrane. It is reported [7] that the locations most susceptible to leakage are near the curbs and in the wheel paths, with center lanes having the minimum damage. In locating the electrical resistivity grid patterns (see Section 4) on the test sections, attempts were made to consider the latter and to provide a 5 ft (1.5 m) spacing for data points, as indicated in ASTM D3633.

5.2. Electrical Potential Testing

A full description of electrical potential testing has been published by ASTM (see Appendix F for ASTM C876). Figure 16 illustrates the basic configuration of the electrical circuit of electrical potential testing using copper/copper sulfate half-cells.

Several key points of the testing procedure as reported in ASTM [8] include at least the following:

(A) A direct connection to the reinforcing steel must be made, except in cases where it can be documented that an exposed steel member is directly attached to the reinforcing steel. Electrical continuity of steel components with the reinforcing steel can be established by measuring the resistance

*A permanent connection should be established in each unit by a thermal weld to the top mat.

between widely separated steel components on the deck and reading duplicate test measurements.

(B) If, after prewetting the concrete surface, the measured value of the half-cell potential changes or fluctuates with time continuously, either the electrical resistance of the circuit is too great to obtain valid half-cell readings of the steel, or stray current from a nearby direct current traction system or other fluctuating direct current, such as arc welding, is affecting the readings.

(C) The electrical contact solution is composed of a mixture of 3.2 oz (95 ml) of the wetting agent thoroughly mixed with 5 gal (19 l) of potable water. For temperatures less than about 50 degrees F (10 degrees C), approximately 15% by volume of denatured alcohol must be added to prevent clouding of the electrical contact solution.

(D) Spacing between measurements should generally be reduced where adjacent readings exhibit algebraic reading differences exceeding 150 mv (areas of high corrosion activity).

(E) Correct the half-cell readings for temperature if the half-cell temperature is outside the range of 72 $\pm$ 10 degrees F (22.2 $\pm$ 5.5 degrees C). The cell has a temperature coefficient of about 0.0005 v more negative per degree F for the temperature range from 32 to 120 degrees F (0 to 49 degrees C).

(F) If positive readings are obtained, they generally indicate insufficient moisture in the concrete and should not be considered valid.

It is important to note that the test cannot be performed when the deck is frozen because of high electrical resistance of ice [7]. Although the test can be conducted at temperatures between freezing and 50 degrees F (10 degrees C), it is recommended that the test be performed when both the deck and ambient temperature are above 50 degrees F (10 degrees C).

5.3. Pachometer Survey

The following is a general procedure to conduct a pachometer survey reported in NCHRP Project "Durability of Concrete Bridge Decks" [7].

A pachometer is generally operated in accordance with the manufacturer's recommended procedure. In operating a hand-held pachometer, the long axis of the probe is oriented parallel to the uppermost reinforcing bar at the point of measurement, and the average depth of cover of the two bars on either side of the point is usually recorded by moving the probe at right angles until the meter pointer indicates a maximum deflection. If the orientation of the top mat reinforcing steel is not known at the time of testing, the probe is rotated at several locations until a sharply defined maximum deflection (minimum reading) is obtained. In this case, the probe is exactly above a bar and the long axis of the probe coincides with the orientation of the bar. In pachometer surveys, it should be kept in mind that measured values may be less than actual depths of cover, depending on existence of particles of magnetic iron in the concrete. Thus, a correction factor should be established. This could be done by coring and measuring the difference between recorded and actual value.

5.4. Core Sampling

Two different types of core sampling are required in this study: first, core sampling through AC and membrane; and second, core sampling through concrete up to the level of top mat reinforcing steel. When core sampling through AC and membrane, dry sawing should be employed to obtain information about presence of water on the deck surface. The core diameter should be 4 in (10.2 cm) to make visual examination of the membrane possible and also to avoid inducing fractures in the concrete during the coring operation of concrete. Examination of core samples will be done in the WSDOT Materials Laboratory. This includes the condition of membrane, such as thickness, bond to asphalt and concrete, and appearance, which should be noted in the core log (see Figure 8). Nevertheless, some data should be obtained in the field, such as moisture between membrane and asphalt, membrane and concrete, and rusting of reinforcing bars in the case of concrete cores.

Examinations of core samples are generally subjective and depend on the examiner's opinion. In determining the membrane bond to concrete and wearing surface, the following procedure, adapted from a study conducted by the New York State Department of Transportation [3], could be used:

In both instances, the bond is considered "good" if the membrane could not be removed by hand or easily with a small spatula, "fair" if the membrane could be removed partially or totally by hand or with a small spatula, and "poor" if there was no adhesion to the concrete or wearing surface (see Figure 9).

In coring through AC and membrane, if no concrete portion is included it may be possible to determine one bond (i.e., either membrane bond to AC or membrane bond to concrete), in terms of pounds by pulling the core sample out of the hole in the field and the other bond by using the "spatula method" described above.

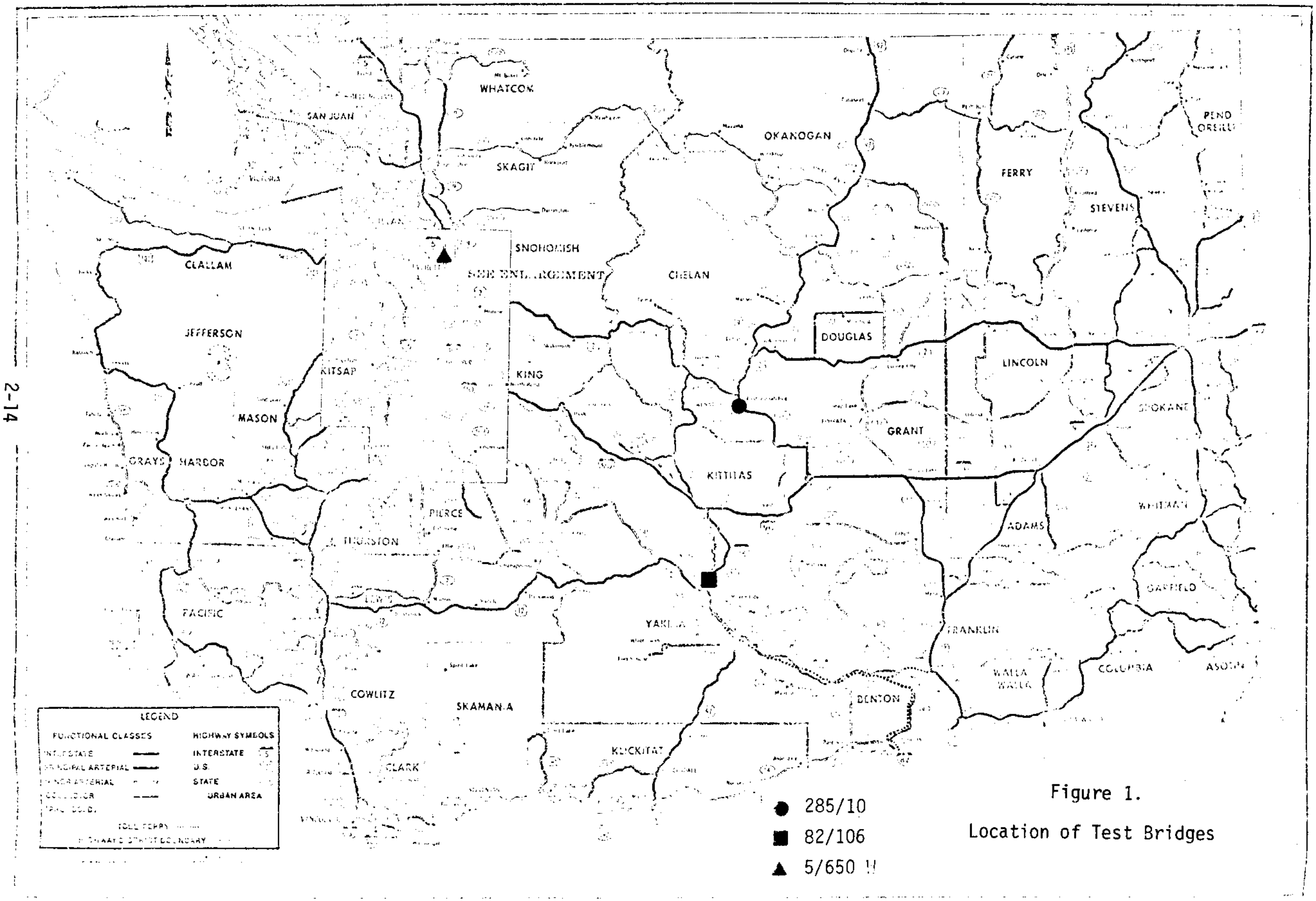


Figure 1.

Location of Test Bridges

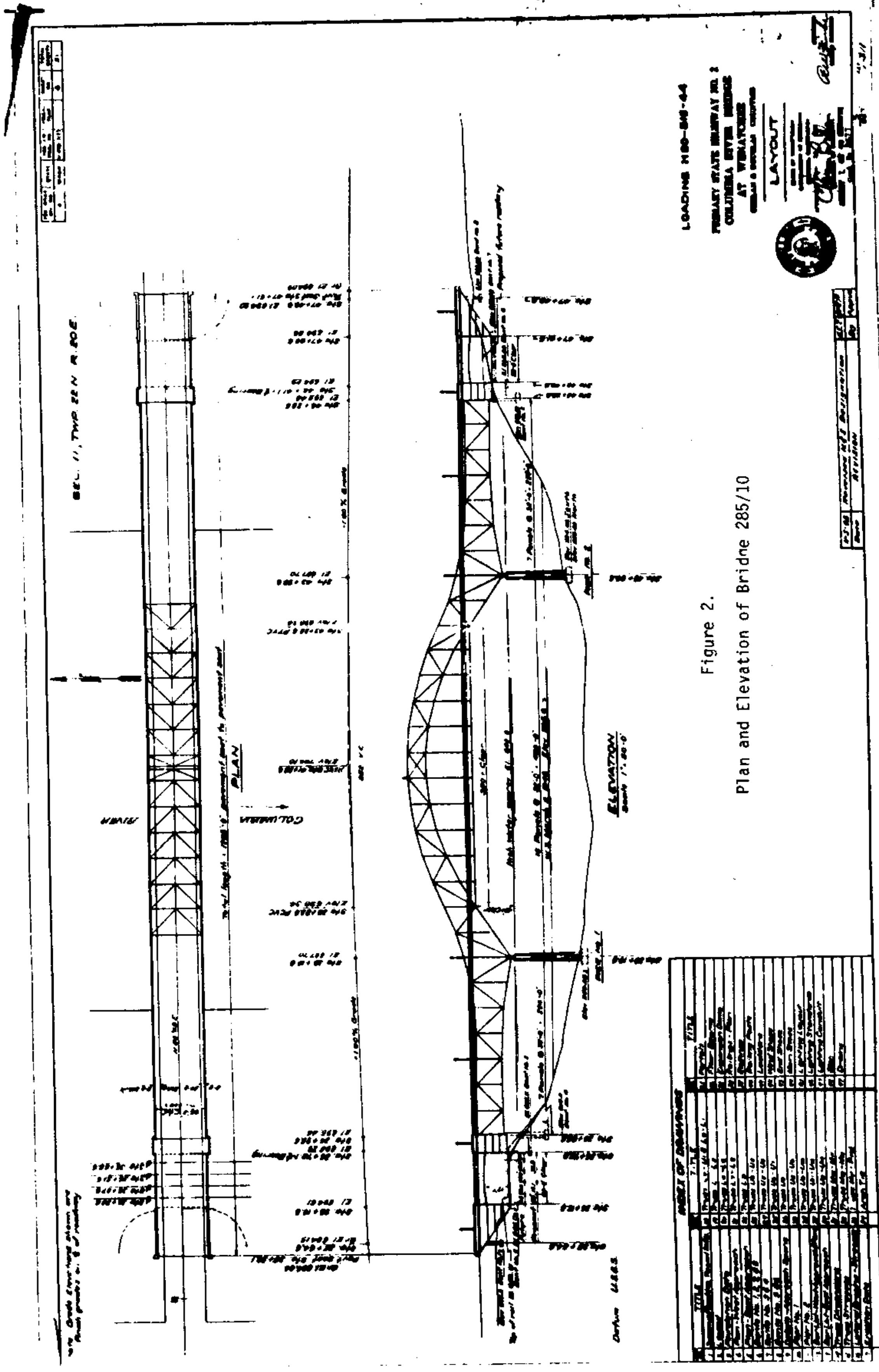


Figure 2.
Plan and Elevation of Bridne 285/10

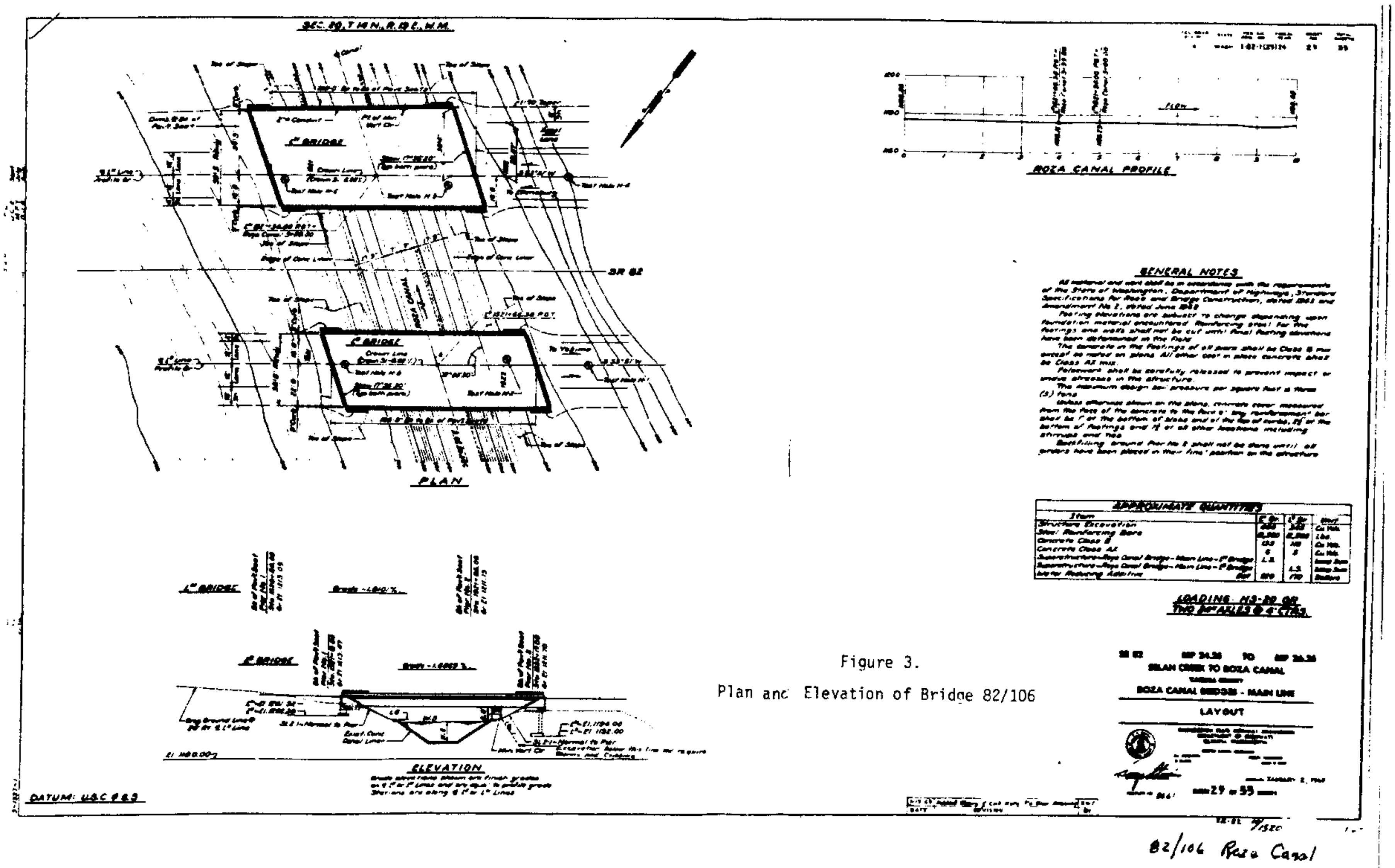


Figure 3.

Plan and Elevation of Bridge 82/106

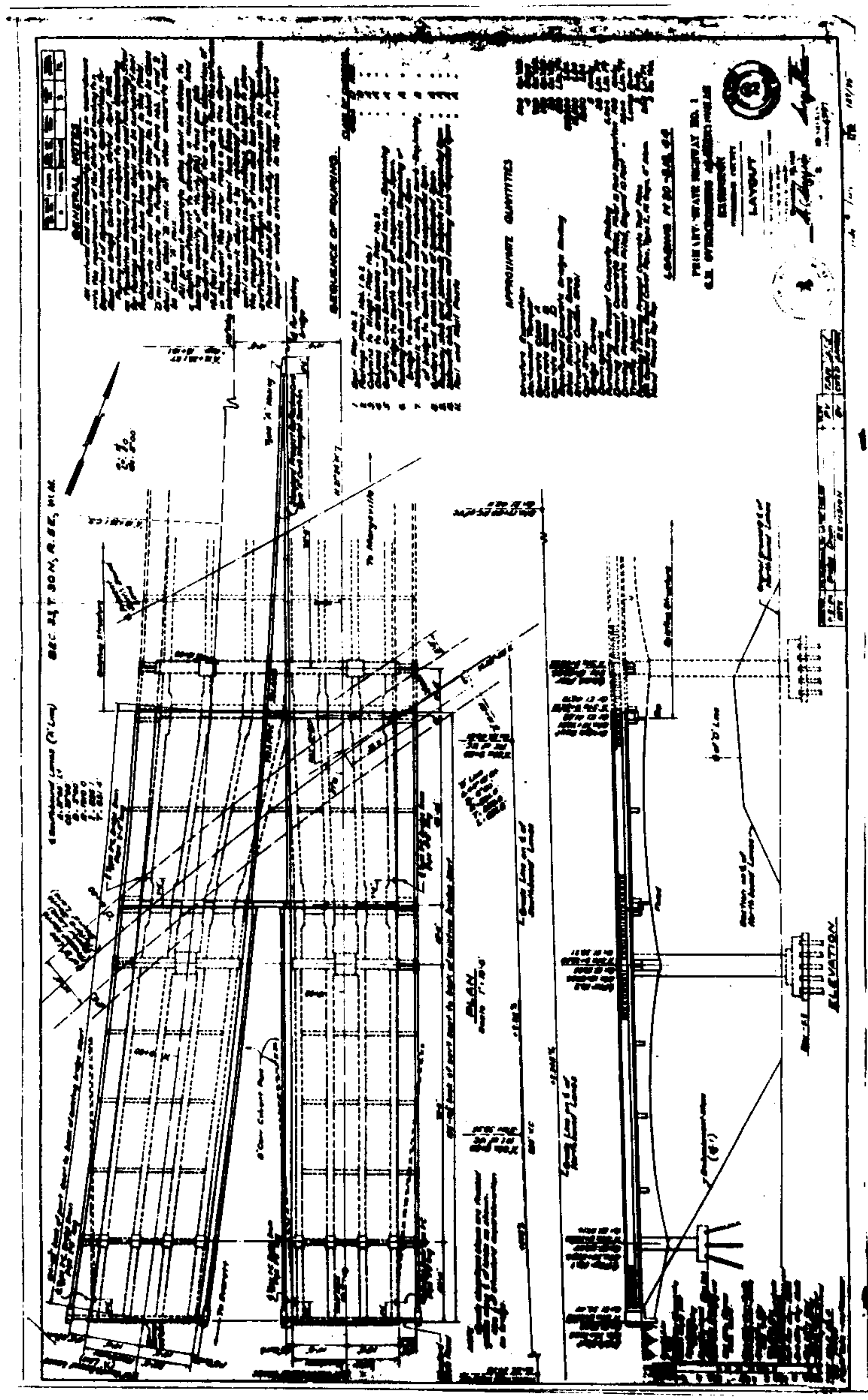


Figure 4. Plan and Elevation of Bridge 5/650 W

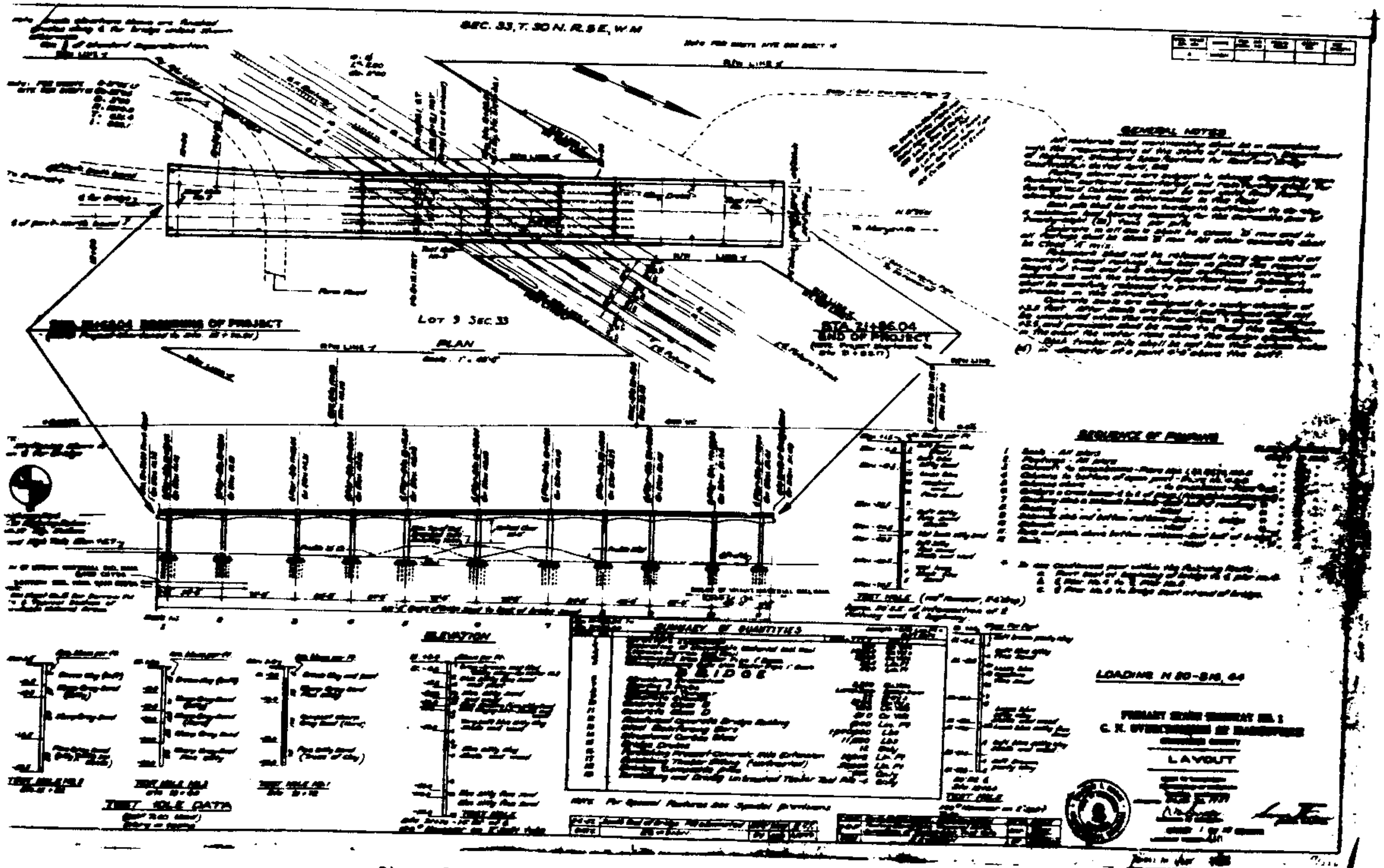


Figure 5. Plan and Elevation of Bridge 5/650 W

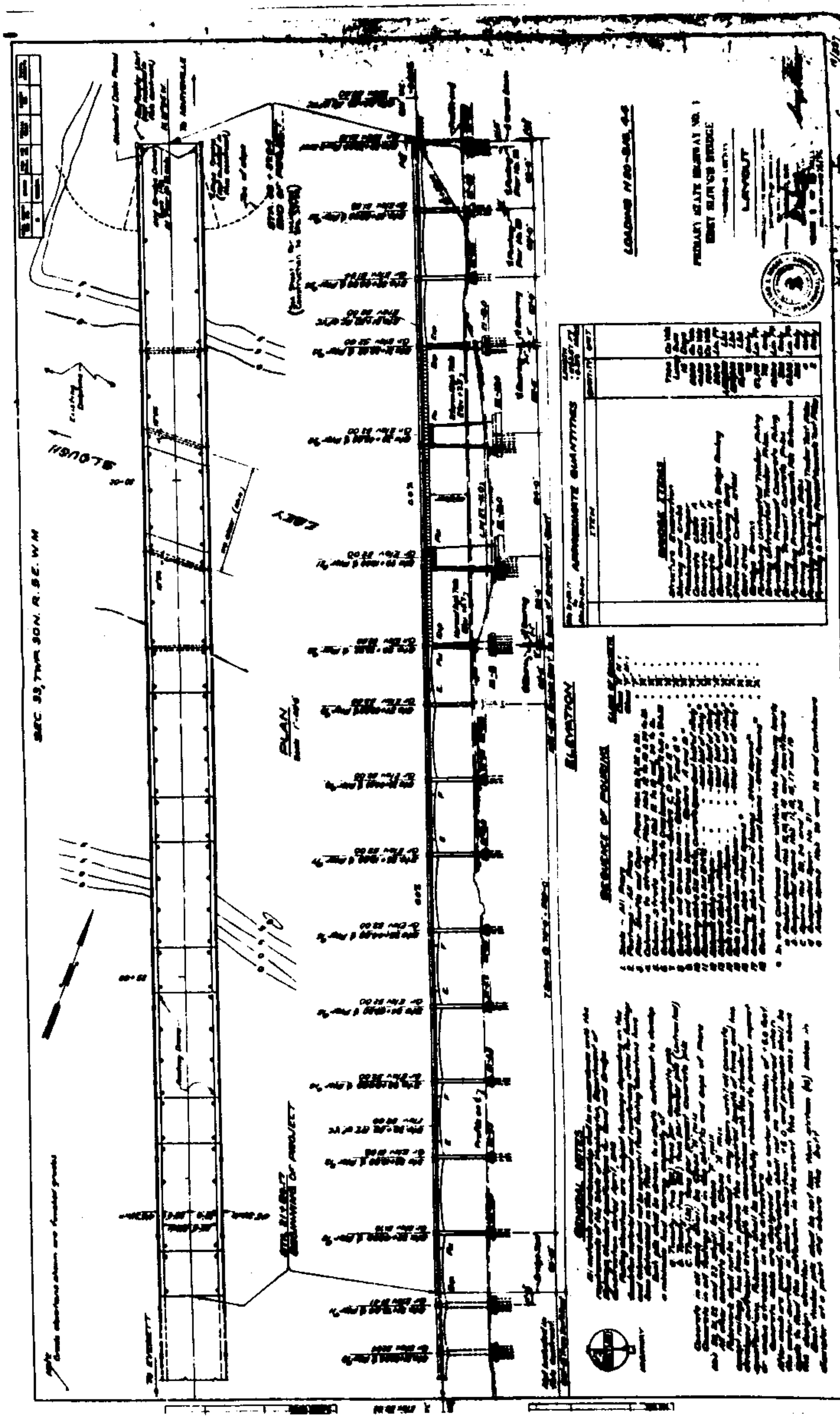


Figure 6. Plan and Elevation of Bridge 5/650 W

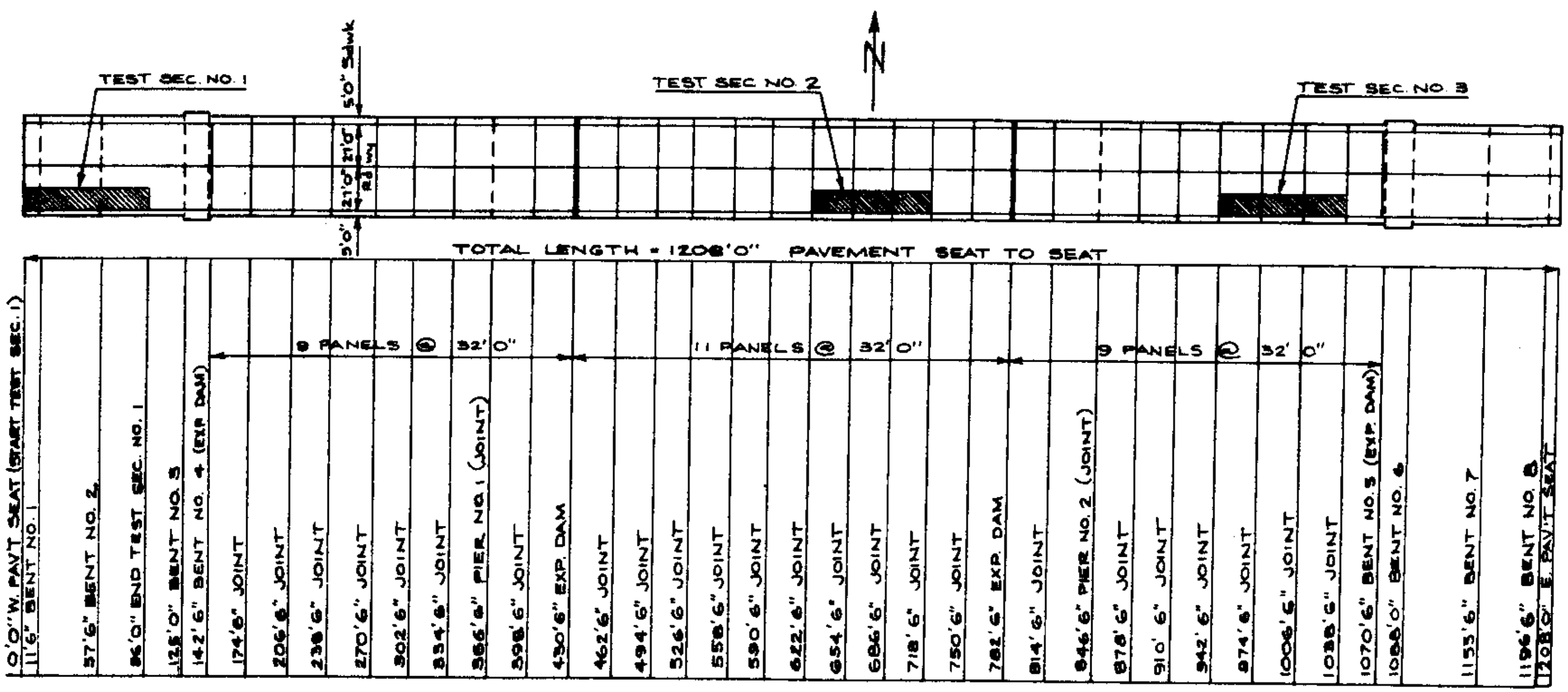
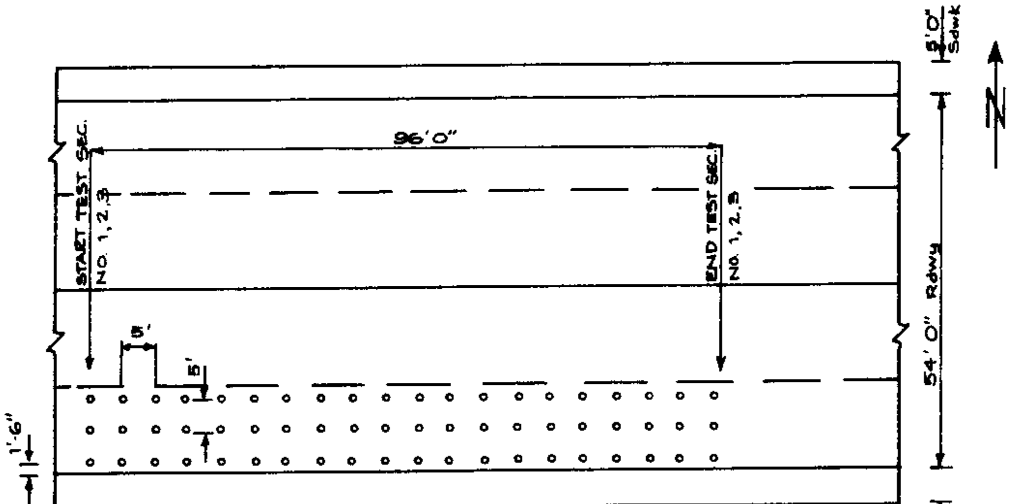


Figure 7.

Layout of Test Section on Bridge 285/10

BRIDGE NO. 285/10
COLUMBIA RIVER BRIDGE
BRIDGE DECK TESTING
(LAYOUT)



GRID PATTERN OF RESISTIVITY AND HALF-CELL TESTING

ACTIVITY	QUANTITY/ TEST SECTION	TOTAL
CHAIN DRAG EXISTING AC (FT²)	(96' × 11.5') = 1104	3312
NO. OF RESISTIVITY READINGS ON EXISTING AC	60	180
NO. OF CORES OF AC AND MEMBRANE	4	12
AREA OF OVERLAY REMOVAL (FT²)	(96' × 11.5') = 1104	3312
CHAIN DRAG CONCRETE (FT²)	(96' × 11.5') = 1104	3312
NO. OF HALF-CELL READINGS ON CONCRETE	60	180
NO. OF CORES OF CONCRETE	4	12
NO. OF DEPTH OF REBAR READING	10	30
NO OF CHLORIDE SAMPLES	8	24
NO OF CHLORIDE DEPTH PROFILES	1	3
NO. OF RESISTIVITY READINGS ON NEW MEMBRANE	60	180
NO. OF RESISTIVITY READINGS ON NEW AC	60	180
CHAIN DRAG NEW AC (FT²)	(96' × 11.5') = 1104	3312

BRIDGE NO. 285/10
COLUMBIA RIVER BRIDGE
BRIDGE DECK TESTING
(GRID PATTERN)

Figure 8.

Electrical Resistivity and
Electrical Potential Grid Pattern
for Bridge 285/10

			None	Slight	Moderate	Severe	Good	Fair	Poor	Remarks
Interface	Membrane to Asphalt	Bond								
		Moisture								
	Membrane to Concrete	Bond								
		Moisture								
Membrane Appearance		Cracking								
		Brittleness								
		Thickness Uniformity								
		Pinholes								

Figure 9. Data Form for AC and Membrane Core Samples

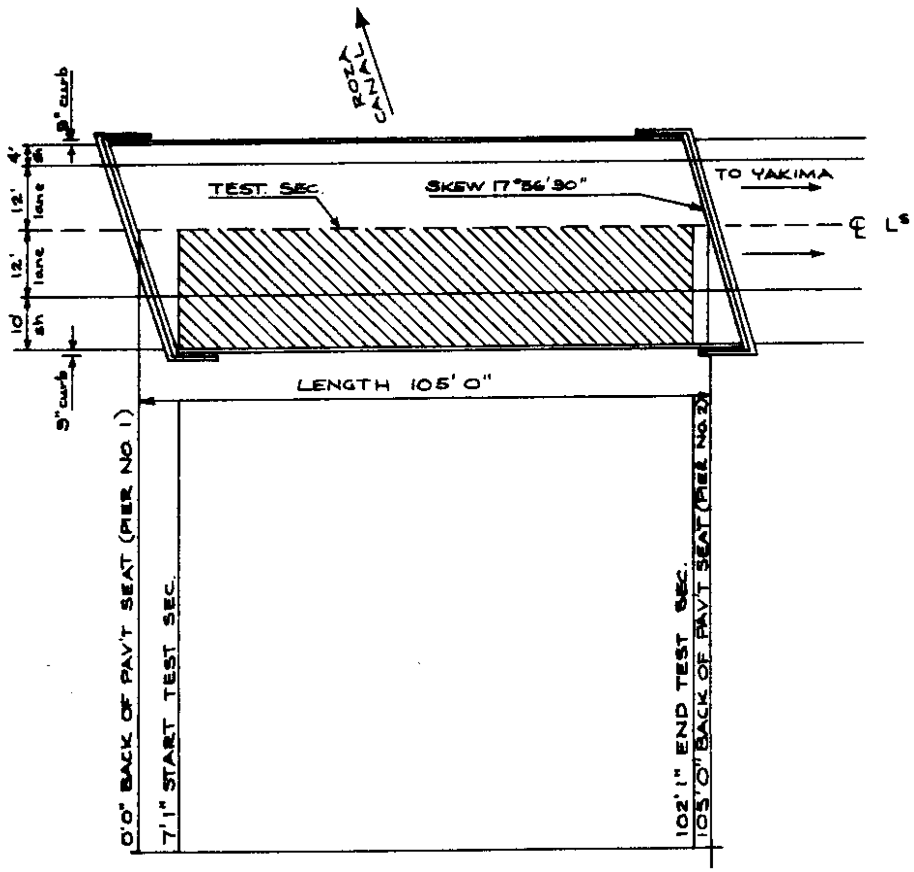
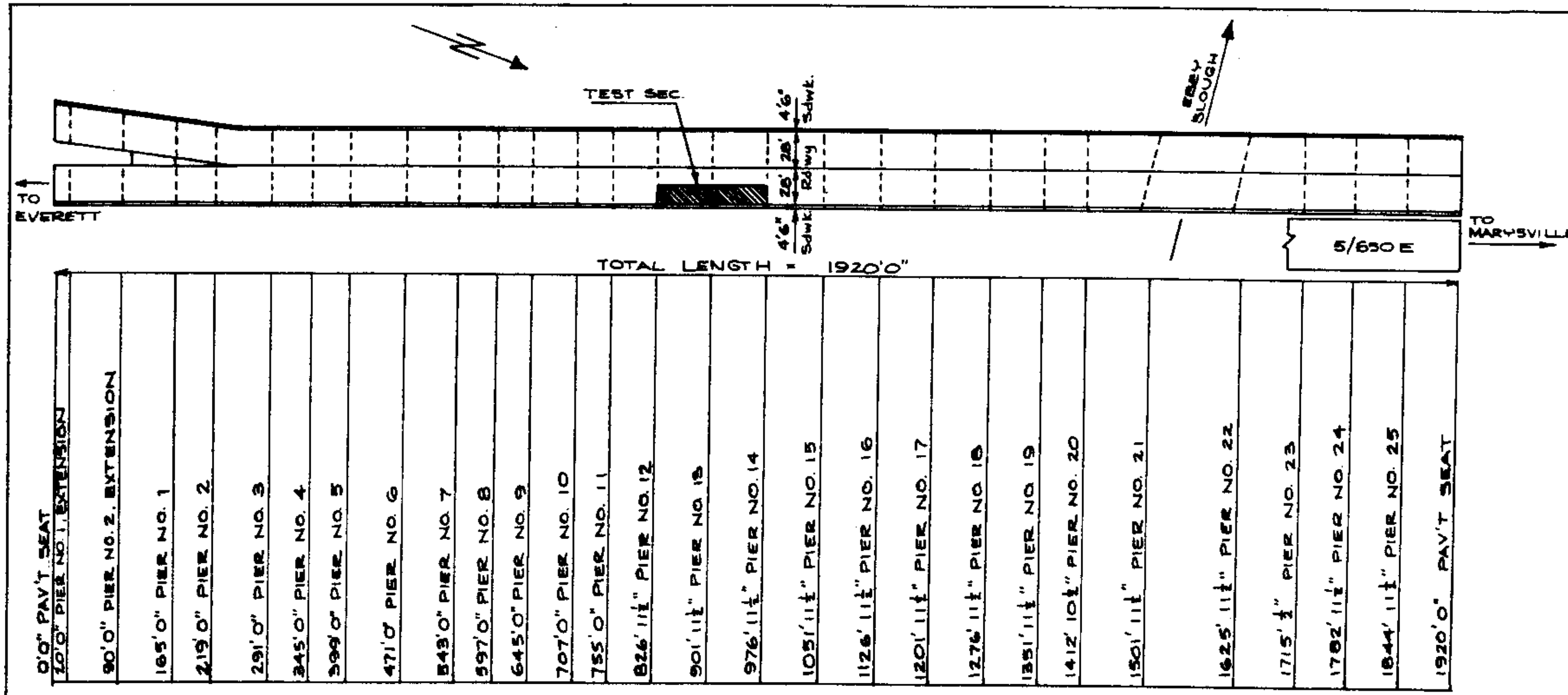


Figure 10.

Layout of Test Section on Bridne 82/106 L/S

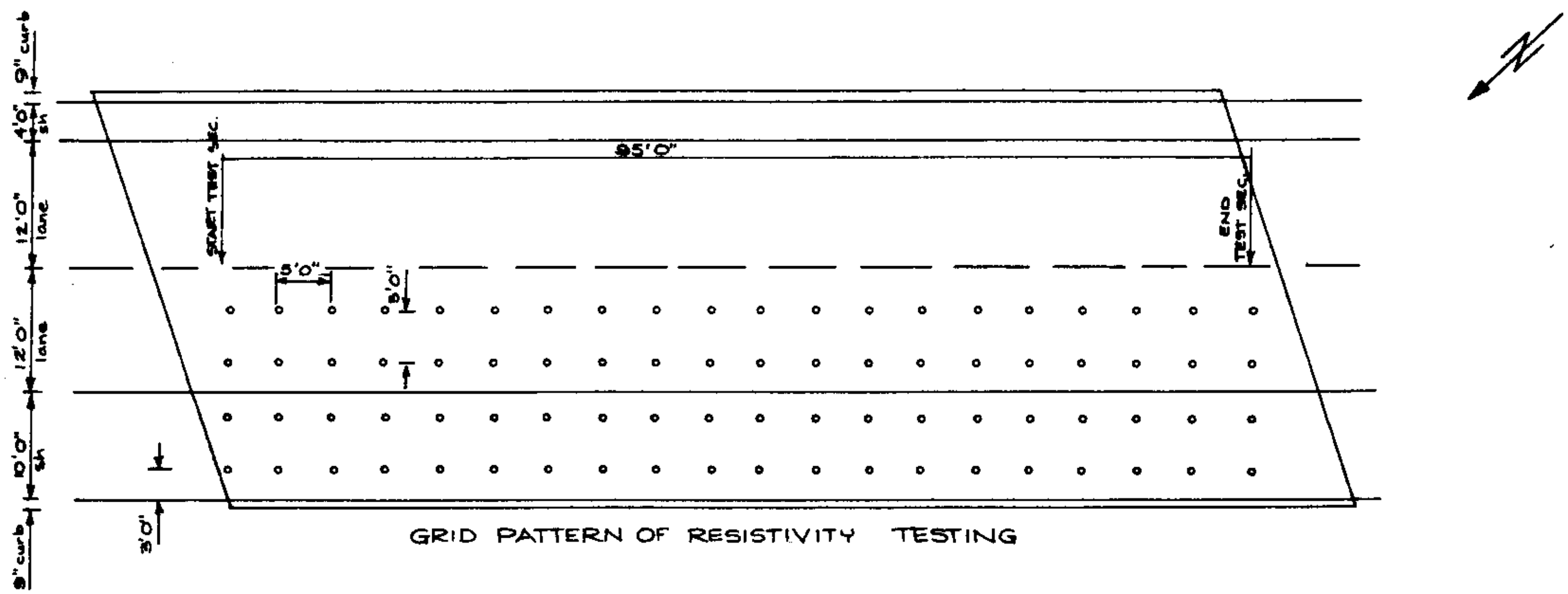
BRIDGE NO 82/106 (L^s)
ROZA CANAL BRIDGE
BRIDGE DECK TESTING
(LAYOUT)



BRIDGE NO. 5/650W
EBEY SLOUGH BRIDGE
BRIDGE DECK TESTING
(LAYOUT)

Figure 11.

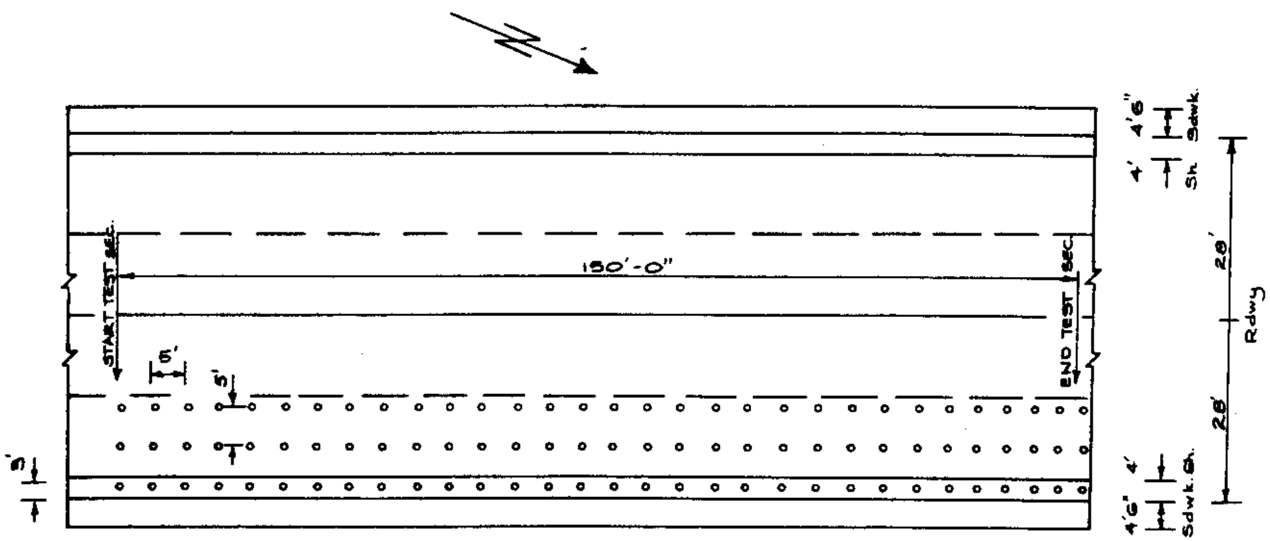
Layout of Test Section on Bridge 5/650 W



ACTIVITY	QUANTITY
AREA OF TEST SECTION (FT²)	(95'×18')=1710
NO. OF RESISTIVITY READINGS ON AC	80
NO. OF CORES OF AC AND MEMBRANE	15
NO. OF HALF-CELL READINGS ON CONCRETE	15
NO. OF CHLORIDE SAMPLES	15
NO. OF CHLORIDE-DEPTH PROFILES	2

Figure 12. Electrical Resistivity Grid Pattern for Bridge 82/106 L/S

BRIDGE NO. 82/106 (L^s)
ROZA CANAL BRIDGE
BRIDGE DECK TESTING
(GRID PATTERN)



GRID PATTERN OF RESISTIVITY TESTING

ACTIVITY	QUANTITY
AREA OF TEST SECTION (FT²)	(150' × 13') = 1950
NO. OF RESISTIVITY READINGS ON AC	93
NO OF CORES OF AC & MEMBRANE	15
NO. OF HALF-CELL READINGS ON CONCRETE	15
NO. OF CHLORIDE SAMPLES ON TEST SEC.	15
NO. OF CHLORIDE SAMPLES ON ENTIRE DECK	5

Figure 13. Electrical Resistivity Grid Pattern for Bridge 5/650 W

BRIDGE NO. 5/650W
EBEY SLOUGH BRIDGE
BRIDGE DECK TESTING
(GRID PATTERN)

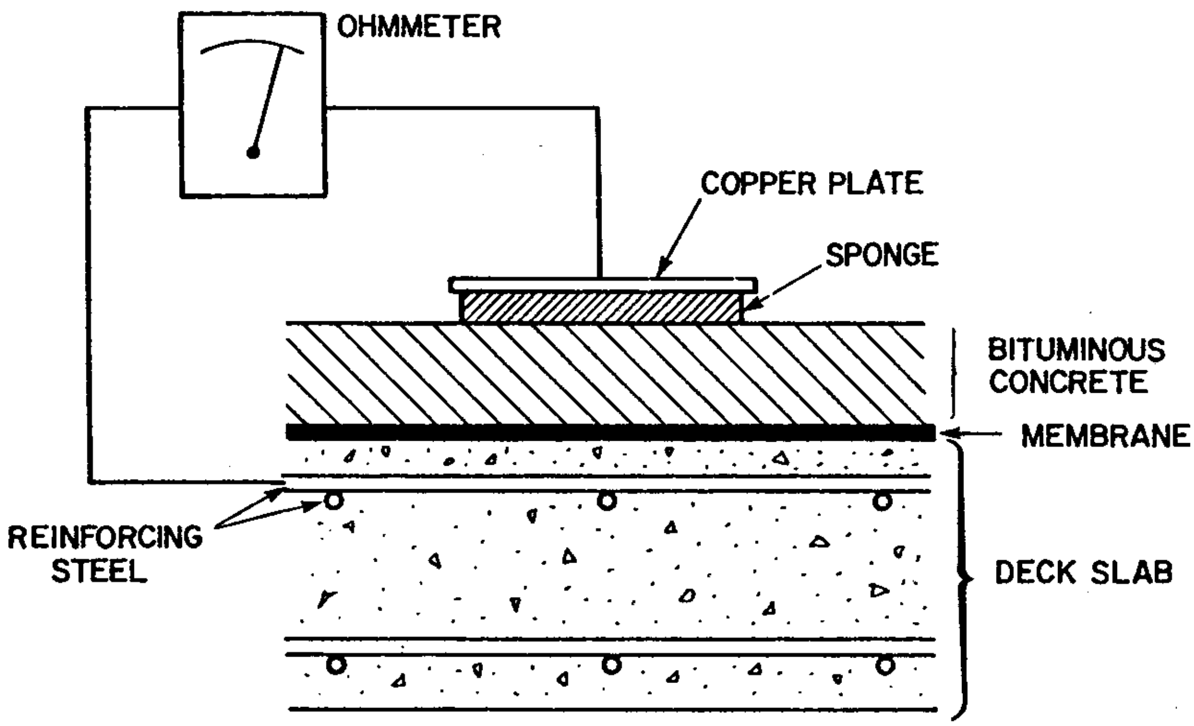


Figure 14. Electrical Circuit for Measurement of Resistance of Deck Membranes [7]

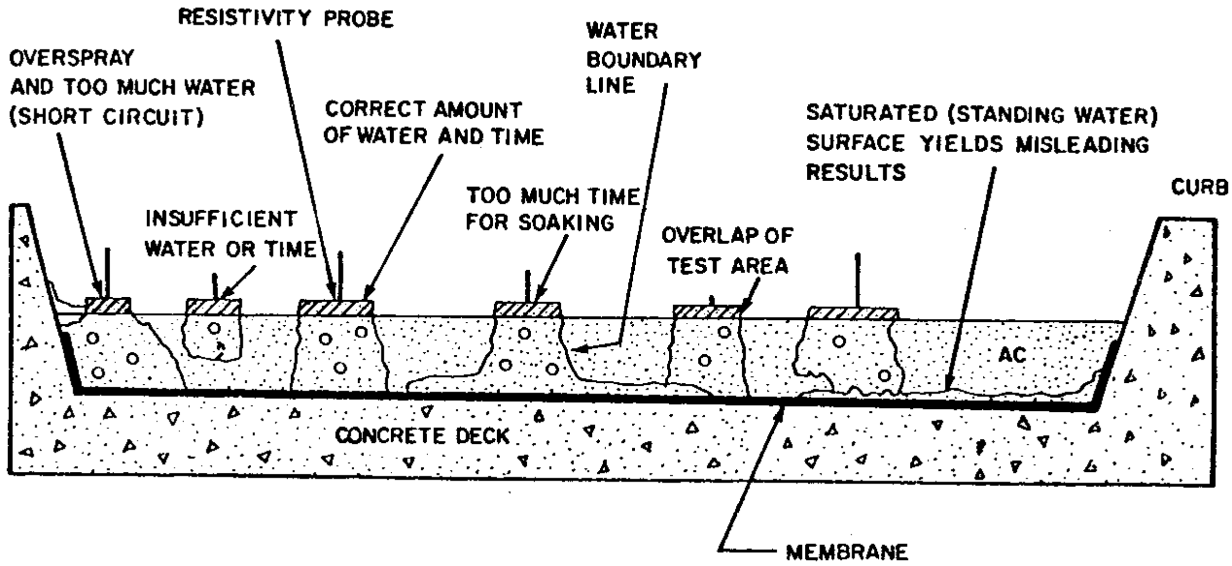


Figure 15. Possible Problems in Resistivity Measurements [7]

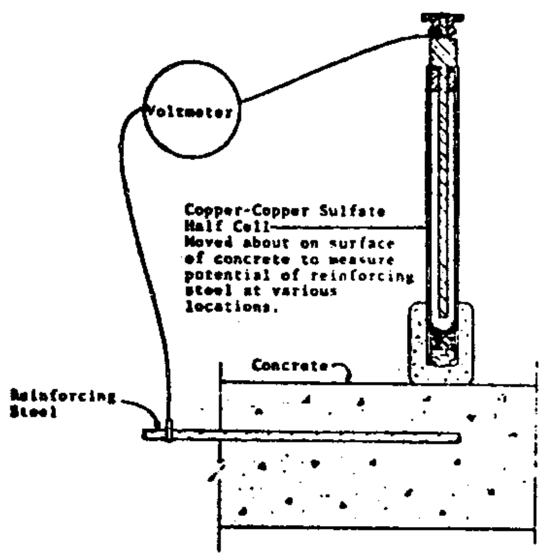


Figure 16. Electrical Circuit for Measurement of Electrical Potential of Deck Reinforcing Steel Using Copper/Copper Sulfate Half-Cell [8]

Table 1. Summary of Data on Test Bridges

Bridge No. and Name		5/650 Ebey Slough	285/10 Columbia River	82/106 Roza Canal
Year built		1954	1950	1969
Concrete of Slab	Cover thickness (in)	1.5	1.5	1.5
	Class	A	A	AX
	W/C ratio	0.45	0.45	0.44
	Slump (in)			
	Cement content (lb/c.y.)	610	610	660
	Compressive st. (psi) (28d)	3,600	3,600	4,000
Test Prior to Waterproofing	Chloride content	yes+	yes	no
	Delamination	no	yes	no
	Half-cell	no	yes	no
Type of repair prior to waterproofing		no major repairs	repair of delaminations	new bridge
Year waterproofed		1973	1976	1969
Type of membrane		Fabric	rubberized asphalt	Fabric
AC Surfacing	Class	B	B	B
	Thickness (in)	1.8	1.8	1.5
Grade (%)		0 to 2.248	1.00	1.6869
Ave. daily traffic		16,800	27,000	6,707
Ave. annual salt application		---	6	10
Ave. ann. railfall (in)		35	10	9
Climate*		western	dry eastern	dry eastern

+tested after waterproofing in 1974

*western, cold mountain, dry eastern

Table 2. Summary of Results of Detailed Condition Survey on Bridge 285/10 (1975)

Deck Appearance			Chloride Ion Analysis						Half-Cell Potentials			Delaminations
Spalling and Scaling		Exposed Rebar	Samples Tested	> 1.3 lb/cy	≤ 1.3 ≥ 0.6 lb/cy	< 0.6 lb/cy	High Cl^- Value	Ave. Cl^- Value	High Value (v)	Ave. Value (v)	% above 0.25v	% of Deck area
% of Deck Area	Depth (in)											
1	½ to 1	yes	4	3	1	0	3.45	1.91	0.63	0.42	92.8	6.4

Table 3. WSDOT Inspection Report for Bridge 285/10

Bridge Component	DATE 7/10/78	6/25/79	9/2/80	7/27/81	5/10/82
Deck	Many transverse leaching cracks, most in concrete T-beam and odd panels (1,3,5) of steel arch from each end.	Many transverse cracks, especially in concrete T-beam.	Many transverse leaching cracks.	Transverse leaching cracks in slab.	Transverse leaching cracks in slab.
Surfacing	AC, some wear, generally good.	AC, reflection cracks, wearing.	AC, wearing in wheel lines, some reflection cracks.	AC, wearing in wheel lines, aggregate ravelling at two points.	AC, worn and ravelling in wheel lines.
Curbs	Many nicks, scrapes and spalls from traffic.	Nicks and spalls throughout.	Numerous nicks and spalls.	Spalled along edge.	Spalled and scarred.
Drains	Blocked with grass.	Plugged.	Most plugged.	Plugged.	Blocked.

Table 4. WSDOT Inspection Report for Bridge 82/106 L/S

Bridge Component	DATE 9/24/80	6/16/81	7/20/82
Deck	A few short leaching cracks at southeast.	A few short leaching cracks at southeast.	A few short leaching cracks at southeast.
Surfacing	AC, wearing in wheel lines.	AC, wearing in wheel lines.	AC, wearing in wheel lines, cracked down center line.
Curbs	Some vertical cracking.	Cracked and spalled.	Scraped.
Drains	----	----	----

Table 5. WSDOT Inspection Report for Bridge 82/106 L/N

Bridge Component	DATE 9/24/80	6/16/81	7/20/82
Deck	----	----	----
Surfacing	AC, showing wear in wheel lines.	AC, showing wear in wheel lines.	AC, wearing in wheel lines.
Curbs	Some cracks.	Some minor scrapes.	Minor scrapes.
Drains	----	----	----

Table 6. Summary of Results of Condition Survey on Bridge 5/650 W (1974)

Deck Deterioration		Chloride Ion Analysis					
% of Deck Area	Depth (in)	Samples Tested	> 1.3 lb/cy	≥ 0.6 ≤ 1.3 lb/cy	< 0.6 lb/cy	High Cl^- Value	Ave. Cl^- Value
30	½ to 1	5	2	3	---	2.31	1.51

Table 7. WSDOT Inspection Report for Bridge 5/650 W

Bridge Component	DATE 8/14/79	6/11/80	8/7/81	8/17/82
Deck	Many transverse leaching cracks, some leaching at fillets with dia-phragm and concrete T-beams.	Numerous transverse leaching cracks.	Many transverse leaching cracks.	Many transverse leaching cracks.
Surfacing	AC, rutted in wheel lines, cracked at abutments.	AC, worn in wheel lines, several large areas of grout patching.	AC, heavily worn in wheel lines, many patched areas, some patches breaking out.	AC, worn heavily in wheel lines, many patches with some breaking out again.
Curbs	Some vertical crack-ing, some traffic abrasion and spalls.	Some vertical cracks, some spalls.	Scraped with some spalls on the edges.	Scraped and spalled.
Drains	Some blocked.	Half plugged.	Some plugged.	Mostly open.

APPENDIX A

PREVIOUS RESEARCH

TABLE OF CONTENTS

1. NEW YORK EXPERIENCE (summarized from [3], published 1977)

In 1970, considerable interest arose within the New York State Department of Transportation to reassess the performance of a two-component bituminous-epoxy waterproofing membrane which had been widely used for deck rehabilitation from 1958 through 1974, and to evaluate the newer membranes then coming on the market, as listed below:

- asphalt-fiberglass
- plasticized neoprene-hypocon (Durok-Rubber coat)
- heat-modified bituminous-resin sheet (Royston Bridge Membrane)
- hot bituminous mix with scrap rubber (Flo-Mix)
- PVC-coal tar and asbestos roofing felt (Posh NEA 4000)
- modified polyurethane-elastomer (Polytok 165)
- EPDM-rubber sheeting, unvolcanized (Nordel)
- liquid polyurethane (Bestaseal)
- cutback-coal tar epoxy resin (Pennetrant)
- hot-mixed rubberized coal tar (Nexus)

This called for a more systematic, quantitative evaluation of the membranes than the ambiguous techniques of visual observation of deck leakage used previously. Thus, a study was initiated in 1971 using the newly developed elecrical resistivity and electrical potential testing to measure the permeability of the membranes and to determine the corrosion activity of the reinforcing steel, respectively.

1.1. Field Testing

(A) Bituminous-Epoxy Membrane: Thirty existing bridge decks were selected for evaluation of bituminous-epoxy membrane. Generally, only one set of electrical resistance and electrical potential measurements were taken. The membranes ranged in age from one month to five years at the time of measurement and all were placed during rehabilitation of existing decks after excavating and repairing the unsound concrete. In addition, 4-in diameter cores were drilled from preselected sites to clarify the condition of the membrane, concrete, and steel in association with various levels of electrical potential and resistance. Each core was then examined visually and the following characteristics noted:

- membrane appearance
- membrane thickness
- concrete appearance
- rebar corrosion
- chloride content at the level of reinforcing steel

(B) Experimental Membranes: Twenty-three sites on 14 existing decks were selected for the evaluation of the experimental membranes. Each experimental membrane was placed on the surface of the deck from which the old wearing course had been removed, together with the old membrane, if present, and from which unsound concrete had been excavated. Subsequently, electrical potential and resistance measurements on the experimental membrane systems were conducted at different times. Thus, each installation had more than one set of measurements. Generally, the installations were not designed to provide definitive answers regarding the behavior of the membranes, and they were regarded as a series of independent case experiments.

1.2. Results

The results from this investigation may be summarized as follows:

(A) The condition of concrete and reinforcing steel in the core samples was in agreement with the interpretation of electrical potential values taken from the same site. In other words, core samples associated with electrical potential values greater than 0.35 v genrally had a higher frequency of cracked or disintegrated concrete and rusted rebars than those associated with potentials less than 0.20 v.

(B) The condition of membrane in the core samples was in agreement with the interpretations of electrical resistance measurement taken from the same site. In other words, core samples associated with electrical resistance values less than 100 KΩ generally had a higher frequency of poorly-appearing membranes, and poor bond to concrete as well as to bituminous wearing surface, than those associated with resistance values greater than 500 KΩ.

(C) Electrical potential values greater than 0.35 v and those smaller than 0.20 v were highly associated with chloride contents greater and smaller than 1.1 lb/cy (0.65 kg/m^3), respectively.

(D) Generally, analyzing the data obtained from the electrical resistance and half-cell potential tests revealed that the corrosion of reinforcing steel had continued in spite of the membrane treatment. The results indicated that the bituminous-epoxy system failed to perform as expected, and the experimental membrane systems as alternatives to the bituminous-epoxy system, as a group, had performed better, but not outstandingly so.

(E) It was noticed that three of the only four experimental membrane installations (three Polytok 165 and one Royston membrane) that consistently performed better than 75% of the bituminous-epoxy installations had been preceded by careful leveling and smoothing of the deck surface. Thus, serious consideration should be given to a requirement for surface smoothness before any membrane is placed on a rehabilitated bridge deck.

2. ILLINOIS EXPERIENCE (summarized from [5], published 1981)

Performance of 20 chloride-contaminated, partially restored, and membrane-waterproofed concrete bridge decks was evaluated in the State of Illinois for three years, starting in 1977. Attempts were made to determine the behavior and service lives of the membrane-bituminous wearing course systems where chloride contents of the decks were over 2.0 lb/cy (1.18 kg/m^3) and reinforcing steel was actively corroding at the time the membrane and bituminous wearing course were constructed.

Illinois concrete bridge deck rehabilitation procedure consists mainly of repair and replacement of spalled and delaminated concrete, followed by application of a primer with a coal-tar waterproofing membrane, adding a sand-asphalt cushion and topping the system with a dense-graded bituminous wearing course. All sound concrete remains in place regardless of the chloride content.

2.1. Field Testing

Generally, field testing included surface condition survey, delamination detection, electrical potential measurement, chloride content measurement, electrical resistivity measurement, and determination of rate of corrosion. The testing was conducted in two phases, namely, preconstruction and postconstruction survey.

2.1.1. Preconstruction Survey

Prior to rehabilitation, corrosion of reinforcing steel was measured by taking electrical potential readings on a 4-ft (1.2 m) grid in the deck. Areas with readings more negative than -0.35 v were considered to be actively corroding.

Surface conditions were mapped to scale and surface defects were located.

Delaminations were located using a Delamtect. Traces were made longitudinally starting from one end of the deck and returning from the other end. The traces were aligned using the 4-ft (1.2 m) grid marks from the electrical potential survey.

Concrete dust samples for chloride analysis were obtaind randomly from undelaminated areas where the electrical potential measurements were less negative than -0.35 v. Ten samples were taken on each deck greater than 300 ft (90.9 m) in length and 5 samples on each deck smaller than 300 ft (90.9 m) in length.

Using the data from the preconstruction survey, the conditions of the decks to be membrane-waterproofed were determined by summation of the "% area corroding" plus "% area with Cl > 2 lb/cy or 1.18 kg/m^3 ." The results indicated that every deck had more than 50% of the concrete chloride contaminated to a level exceeding the threshold level. If the 1974 FHWA guidelines had been followed, complete deck replacement would have been required for an acceptable permanent reconstruction.

2.1.2. Postconstruction Survey

After rehabilitating and waterproofing the decks, surface defects were mapped every year.

Delaminations were located using the Delamtect. Each year the traces were made on 2-ft (0.6 m) centers, with the first trace starting about 1 ft (0.3 m) from the outside curb.

To determine the permeability of the membranes, two methods were used. In the first method, permeability measurements were made using electrical resistance testing on the new surfaces. The second method was measuring the resistance between pairs of copper strips, 4 in (10 cm) apart and 9 ft (2.7 m) long, placed transversely on the bare decks. Three sets of strips were placed on each deck prior to the placement of the penetrating primer. Resistance measurements were conducted annually.

Rate of corrosion was determined in terms of mils (25.4μ) per year of corroding metal. It was assumed that a decrease in the rate of corrosion would be an indication of an effective waterproofing. Two methods were used. In the first method, Corrosometer probes were buried in the deck near the top reinforcement. The measurements were taken frequently each year, and corrosion rate in mils (25.4μ) per year was calculated using the manufacturer's formula. The second technique was based on polarization resistance. Types of installations included 2-electrode and 3-electrode systems (see [5] for more details regarding instrumentation and installation).

2.2. Results

The results of this investigation may be summarized as follows:

(A) Surface Condition Survey: One year after rehabilitation and waterproofing, some hairline cracks were observed in longitudinal construction joints. Dish-shaped depressions were observed in the surfaces of most of the decks. After two years of service, the longitudinal construction joints were cracked. In addition, transverse cracking was observed on most decks. Y-shaped cracks were also observed in dish-shaped depressions.

(B) Delamination Survey: After one year of service, the new decks contained, on the average, 0.4% delaminated area. In the second and third years, the averages were 0.6% and 0.3%, respectively. However, comparison of the strip tapes from the Delamtect indicated that some delaminations disappeared while others had moved. It was concluded that most of the delaminations located on the bituminous wearing course were blisters under the membrane. This was evidenced by observed dish-shaped depressions, the apparent healing of delaminations, and movement of delaminations.

(C) Membrane Permeability Measurements: Results of electrical resistance testing showed good initial protection on 15 decks (readings grater than 500 KΩ) and 3-6% doubtful areas on the other 5 (readings between 500 KΩ and 100 KΩ). Results of all resistance measurements made on the copper strips showed only one deck with questionable protection (readings greater than 10 KΩ represented good protection while any lesser value was considered questionable). After three years of installation, only 70% of the pairs of copper strips were still in service, since the lead wires were lost due to snowplow damage and could not be located.

(D) Rate of Corrosion Measurements: Data from Corrosometer probes indicated that the corrosion rate decreased with time at each installation. After three years of service, most probes appeared to be totally inactive. Polarization resistance installations, in general, showed little activity before the completion of waterproofing and less after completion. However, no definite conclusion could be reached based on the collected data, since no information was available on the quantitative accuracy of the measurements.

(E) Conclusion: Regarding the entire testing program, it was concluded that the original goal of the study, which was to determine if waterproofing the surface of the concrete decks containing chloride in excess of the corrosion threshold would control future deterioration, could not be attained at that time. Thus, it was recommended that another 5 to 7 years of study be undertaken to continue the data collection analysis phase to assess the performance trends.

3. LOUISIANA EXPERIENCE (summarized from [9], published 1978)

In an attempt to evaluate the effectiveness of waterproofing membrane systems applied on concrete bridge decks, the Louisiana Department of Transportation started a study in 1975 which included research evaluation of experimental installations on the selected sites as well as laboratory evaluation of the membrane systems. The following is a listing and description of the six membrane systems included in this testing program:

System No. 1 (Superseal 4000) - a high or low heat, hot poured material from a double boiler, single component hot applied elastomeric PVC polymer membrane used with 65 lb roll roofing paper (ASTM 224) as a part of the system.

System No. 2 (Hydro-Ban RVN-30) - a reinforced vinyl-neoprene or vinyl-butyl elasotmeric waterproof membrane used with an adhesive/ primer, a spray topping and tape weld as a part of the system.

System No. 3 (Protecto Wrap) - a cold applied primer, membrane, mastic and protection sheet. The membrane is a laminate of premium grade aromatic tars modified with synthetic resins and reinforced with a synthetic non-woven fabric.

System No. 4 (Heavy Duty Bituthene) - a high-strength, heat-resistant mesh embedded between a self-adhesive rubberized asphalt and non-tacky bituminous compound. A primer and a mastic are part of the system.

System No. 5 - (Hydro-Ban RVN-45) - a reinforced vinyl-neoprene or vinyl-butyl elastomeric waterproof membrane used with an adhesive/primer, a spray topping and tape weld as a part of the system.

System No. 6 (Royston) - a prefabricated laminate consisting of an impregnated fiberglass sandwiched between layers of bituminous mastic with a top surface of polyester film used with a primer as a part of the system.

Figure A-1 gives the span location of the various waterproofing membrane systems on the selected sites.

3.1. Laboratory Testing

This included electrical resistance test, tension test, bend test, and heat-aged bend test conducted on the membranes as described below:

(A) Electrical Resistance Test: After the membrane was applied to a 2

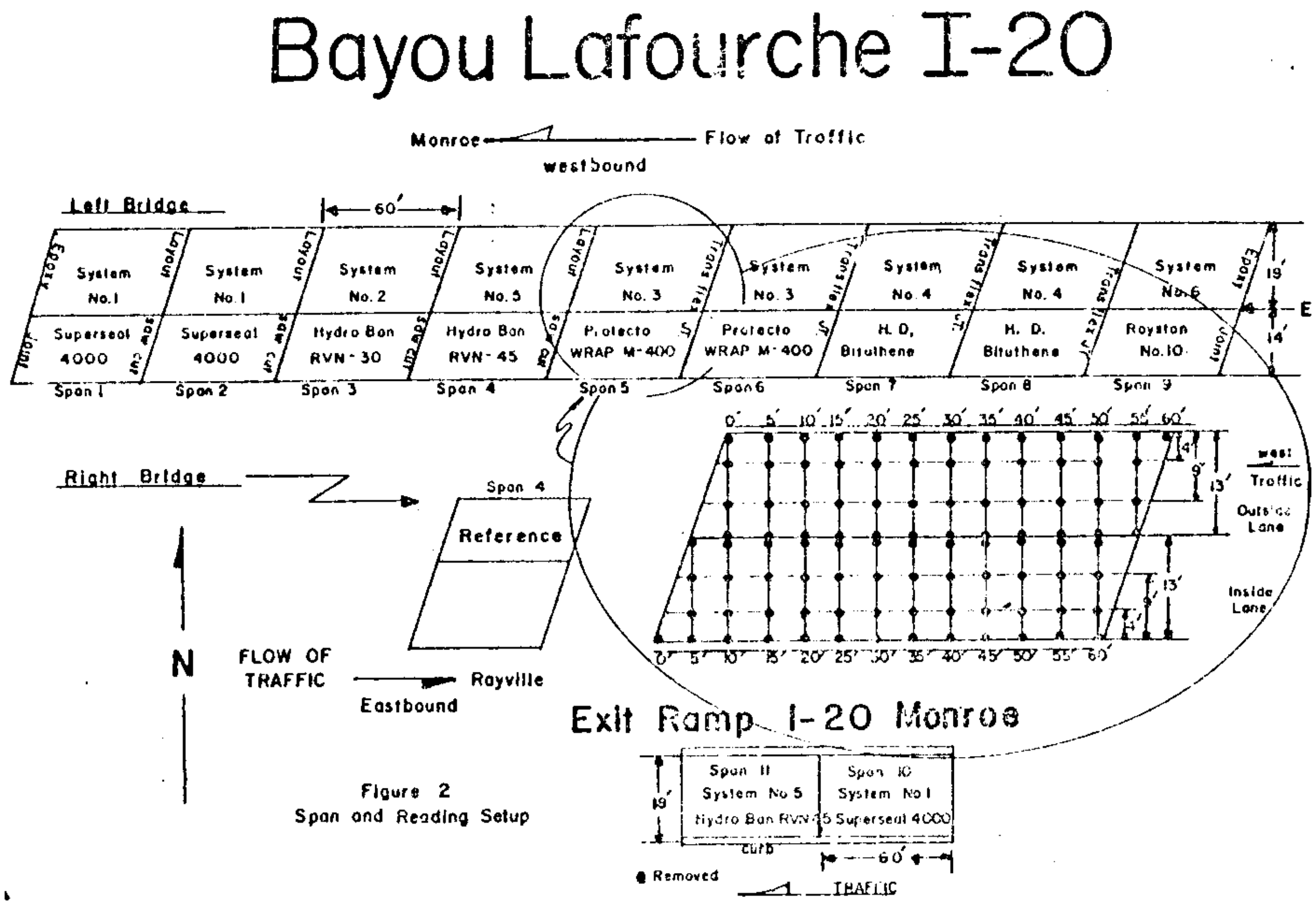


Figure A-1. Span Location of the Various Membranes on the Selected Sites [9] x 6 x 12 in (5 x 15 x 30 cm) concrete block, the initial resistivity of the membrane was determined at room temperature. The final resistivity was determined at -5, 0, 5 and 10 degrees F (-21, -18, -15 and -12 degrees C) after the tension test was conducted.

(B) Tension Test: A membrane was applied on a test concrete block attached to a device which elongated, causing the block to crack in tension. The crack was allowed to open 1/10 in (2.5 mm). Tests were performed at -5, 0, 5 and 10 degrees F (-21, -18, -15 and -12 degrees C).

(C) Bend Test: This test involved bending a 1 x 8 in (2.5 x 20.3 cm) strip of membrane material around a 1 in (2.5 cm) diameter mandrel at the temperatures mentioned previously.

(D) Heat-aged Bend Test: This test was the same as the bend test except the membranes were placed in an oven for 28 days at 140 degrees F (60 degrees C) prior to the test.

3.2. Field Testing

Field testing consisted mainly of electrical resistivity readings of the membranes and visual observations. Resistivity testing was conducted at three different stages as follows:

1. Upon completion of the waterproofing membrane systems application, but prior to the asphaltic concrete overlay.

2. After completion of asphaltic concrete overlay to determine if any damage to the membrane had occurred because of the construction.

3. At intervals of time after the original installation when weather and time conditions were favorable.

3.3. Results

The results of this study may be summarized as follows:

(A) Regarding the laboratory evaluation phase, systems 1 and 6 failed the bend test and heat-aged test. Systems 2, 3, 4 and 5 generally passed the tests and looked promising for field tests.

(B) Regarding visual observation of membrane performance, severe overlay shoving occurred with system 4 on some experimental sites. Systems 2 and 5 showed bonding trouble between membrane and overlay in some locations.

(C) Using field electrical resistivity readings as the chief test for waterproofing ability, all systems evaluated in the field failed to achieve total impermeability of the concrete deck (acceptance level 250,000 ohms) at the last field evaluation and test period. Previous test periods also had indicated the same results, as far as resistivity readings were concerned.

(D) A recommendation was made that the State of Louisiana not accept the waterproofing membrane systems evaluated in the study for use as 100%

impermeable materials on concrete bridge decks.

4. COLORADO EXPERIENCE (summarized from [10], published 1977)

In 1974, the Colorado Department of Transportation established a continuous testing program to evaluate service performance of the membrane waterproofing systems using the electrical resistance test as the sole method. Twenty bridges were selected in the State of Colorado representing a wide range of membrane system types and environmental conditions. The following is a list of the membrane systems applied on the test bridges:

<u>System 1. Built-Up</u>

Included five layers of coal tar and emulsion and two of fiberglass, the layers being placed one at a time. (This system has been phased out due to the difficulty of construction and highly toxic nature of the materials.)

<u>System 2. Prefabricated</u>

Protecto Wrap - a laminate of synthetic resin reinforced coal tar and a non-woven synthetic fibrous mat.

Royston No. 10 - a laminate of an impregnated fiberglass mesh sandwiched between layers of a bituminous mastic.

Heavy Duty Bituthene - a polyethylene film with a rubberized asphalt adhesive on one side.

<u>System 3. Hot Applied Single Component</u>

Superseal 4000 - an elastomeric PVC polymer.

Gilsabine No. 5 - a 25% rubber and 75% asphalt cement with a small amount of extender oil.

Deckseal No. 5 - a 25% rubber and 75% asphalt cement with a small amount of extender oil.

4.1. <u>Results</u>

A summary of the results of the continuous testing program on the membrane systems through five years of service is shown in Table A-1. It was concluded that the most important factor in membrane performance is how well it is placed. Well-constructed membranes of any type, in any type of environment, tested well immediately after placement and continued to test well, whereas poorly placed membranes tested poorly initially and continued to test poorly. It was also noticed that there was a tendency for some membranes to heal with time as indicated by increased resistance readings.

It was decided that future plans should include drilling through the asphalt and membrane to obtain concrete samples for chloride analysis, and to make half-cell measurements.

Table A-1. Results of Five Years of Testing of Membranes in Colorado [10]

Field Dist.	Str. Number	Location	Year Installed	Type or Brand	Roofing Paper	System*	% Success Fall 1974	% Success Spring 1975	% Success Fall 1975	% Success Fall 1976	% Success Fall 1977
I	F-12-U	W. Frisco/10 mi.	1971	Royston #10	NO	2	69	73	77	69	73
I	F-12-V	W. Frisco WB	1971	Royston #10	NO	2	98	97	97	99	95
I	E-17-FE	SH 128 & I 25 EB	1972	Protecto Wrap	NO	2	92	95	96	99	99
I	E-17-HE	SH 128 & I 25 WB	1972	Super Seal 400	NO	3	57	95	95	100	100
I	F-15-CE	El Rancho-I 70	1972	Protecto Wrap	NO	2	99	100	99	100	100
I	F-15-CC	Soda Creek I 70	1972	Superseal 400	NO	3	99	99	99	100	100
I	F-15-CA	Hyland Hills - I 70	1973	Superseal 4000	YES	3	87	94	94	91	90
I	H-17-CG	Larkspur - I 70	1976	Gilsabind No. 5	YES	3	-	-	-	95	96
I	F-12-AN	Vail Pass I 70	1977	Deckseal No. 50	YES	3	-	-	-		100
II	K-18-EP	SH 96 11th St.	1973	Coal Tar/Fiberglass	NO	1	99	100	100	100	100
II	J-16-EI	SH 36 Crows Gulch	1974	Superseal 4000	YES	3	92	88	94	89	92
II	K-18-EJ	SH 45 RR	1974	Protecto Wrap	YES	2	95	99	99	100	100
III	F-10-AB	Walcott W-I 70	1971	Royston #10	NO	2	97	90	96	99	67
III	J-4-AT	Delta W SH 348	1972	Coal Tar/Fiberglass	NO	1	99	91	100	100	100
III	H-3-X	Fruitvale - I 70	1973	HD Bituthene	YES	2	82	85	92	86	99
III	F-3-	Fruitvale - I 70	1973	HD Bituthene	YES	2	-	92	100	100	95
III	H-4-Z	Mesa N & S	1973	Superseal 4000	YES	3	87	-	99	99	92
IV	C-17-EZ	Windsor E	1971	Protecto Wrap	NO	2	82	96	100	96	100
IV	D-24-AO	Sterling	1972	Superseal 4000	NO	3	99	98	99	100	97
IV	D-16-CG	Longmont E.	1972	Superseal 400	NO	3	88	92	96	80	90
IV	E-16-FB	Table Mesa Drive	1973	Superseal 4000	YES	3	57	45	80	35	-
V	O-3-C	Mesa Verde	1972	Protecto Wrap	NO	2	38	69	92	52	-
VI	F-16-DX	8th Ave. I 25	1973	Husky #4	YES	3	-	73	92	0	96
VI	F-16-?C	Sheridan SH 36	1973	Protecto Wrap	YES	2	25	25	40	33	57
VI	F-17-?Z	I 25 & I 225	1977	Gilsabind No. 5	YES	3	-	-	-	-	100
VI	F-17-IG	74th Ave. over Canal	1976	Cabuaro Crumb Rubber	NO	2	-	-	-	-	0

System* 1 Built-up
2 Prefabricated
3 Single Component

5. ONTARIO EXPERIENCE (summarized from [11])

In 1974, a project was approved to evaluate the service performance of waterproofing membranes applied on bridge decks in Ontario. A total of 74 bridges were investigated. The number of bridges investigated may be broken down by type of waterproofing membrane used:

32 bridges - hot applied rubberized asphalt
29 bridges - hot applied rubberized mastic
13 bridges - experimental membranes

5.1. Field Testing

Field testing included resistance tests, corrosion tests, and sampling for chloride analysis, as well as visual examinations.

(A) Resistance Tests: After the deck to be tested was located, ten locations were randomly chosen for resistance tests, and resistance readings were taken.

(B) Corrosion Tests: Adjacent to the resistance test locations and closer to the bridge centerline, a 1/2 in (1.27 cm) diameter hole was drilled through the asphalt and membrane. After the dust in the hole was blown out, it was filled with copper sulfate solution and a half-cell voltage was read both in the hole and on the sponges. The latter readings were usually slightly higher than those recorded in the holes.

(C) Core Sampling: After the electrical resistance and half-cell testing were conducted, 30 of the original 74 decks were revisited, and by means of core sampling, 2-in (5.08 cm) diameter core samples were obtained at the location of the highest and lowest resistance reading and highest and lowest half-cell reading. The core samples were examined visually in the laboratory and the chloride content was determined. In addition to the core sampling, a 1 ft (0.09 m) section of the asphalt overlay was removed by dry sawing, and the condition of the membrane, concrete surface, etc. was noted.

5.2. Results

(A) Resistance Tests: There were indications that the resistance test was not reliable due to the possibility of water in the asphaltic concrete, protection boards, etc., grounding the electrical circuit. It was believed that the migration of the test solution under the sponges could form indeterminate and variable areas (see Figure A-2). This leaking away of the solution could vary with the cross-fall and grade of the deck and it could more readily happen where a protection board, an asphaltic felt board, was used over the membrane. If this board were wet, it would act as a conductor to drains or joints. Nevertheless, assuming the tests gave accurate resistance readings, and considering a membrane with a resistance reading of one or none out of 10 under 400 K ohms satisfactory, the results indicated that 48% and 29% of the decks waterproofed with rubberized mastic and rubberized asphalt, respectively, were performing satisfactorily.

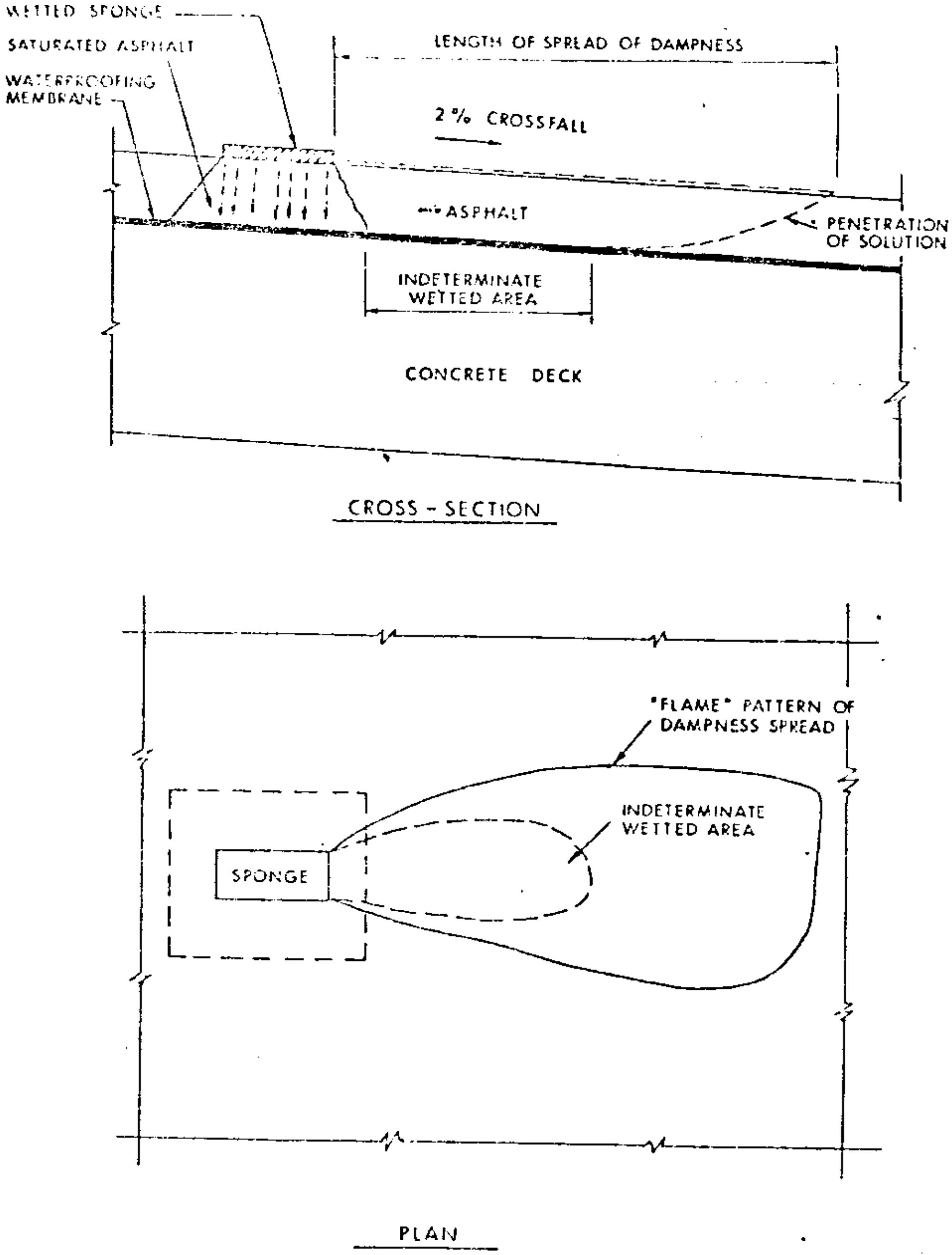


Figure A-2. A Possible Leaking Away of the Electrical Testing Solution [11]

(B) Corrosion Tests: There was no indication that the half-cell test was not accurate. Assuming that bridge decks with no value of voltage greater than 0.35 v* are satisfactory, the membranes rated by percent of bridge decks not corroding were as follows:

- Hot rubberized asphalt decks - 58%
- Hot rubberized mastic decks - 48%

(C) Core Sampling and Visual Analysis: Generally there did not appear to be any correlation between chloride content of the core samples and the corrosion voltage or resistance value in the test sites. Visual analysis of samples sawed from the deck with a dry blade appeared to be the most positive means of evaluating the condition of the membranes, bond to concrete and asphalt overlay, and the surface condition of the deck. The survey indicated that the rubberized mastic tends to be more cracked and segregated, and it hasless elasticity than the rubberized asphalt. Generally, the presence of moisture between the membrane and concrete was the most conclusive evidence of a breakdown in the moisture barrier.

(D) Experimental Membranes: Due to the small number of samples of experimental membranes tested, the relative effectiveness of these systems could not be determined.

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*It would be more appropriate to use half-cell values less negative than 0.20 v to represent non-corroding conditions (authors).

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361658 DA
EPOXY COATED REINFORCEMENT IN BRIDGE DECKS
Wills, J
Transport and Road Research Laboratory Old Wokingham Road Crowthorne RG11 6AU Berkshire England 0305-1315
Trrl
1982 Monograph 20p 4 Fig. 2 Tab. 20 Ref.
REPORT NO.: SR667;
SUBFILE: TRRL; IRRD; HRIS
Some proposals are made for the use of epoxy coated reinforcement in United Kingdom bridge decks. These proposals are based mainly on an extensive review of North American work. A preliminary cost study is made. It is concluded that epoxy coating of the top steel in addition to current waterproofing practice would provide - at relatively little extra cost - additional assurance that the reinforcement would be adequately protected throughout the life of a bridge. The current design rules do not permit decks with permanently exposed concrete wearing surface without waterproofing. Epoxy coating may afford a means of introducing such decks which have certain advantages. However, it is concluded that further information is required before a positive recommendation to delete waterproofing can be made. (A) (TRRL)

349771 DA
BRIDGE DECK CONDITION SURVEY
LaCroix, JE
Illinois Department of Transportation Bureau of Materials & Physical Research, 126 East Ash Street Springfield Illinois 62706 Phy Res 94; Federal Highway Administration 400 7th Street, SW Washington D.C. 20590
Jun 1981 Final Rpt. 42p
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA/IL-094;
CONTRACT NO.: IHR-306; Contract
SUBFILE: HRIS
Performance of 20 chloride-contaminated, partially restored, waterproofed bridge decks was evaluated for three years. Annually, surface conditions were mapped, extent and location of delaminated areas were determined, waterproofing permeability was measured, and attempts were made to determine the rate of corrosion. Insufficient service life performance data were available to determine if sealing surface of partially restored chloride-contaminated decks will affect the rate of deterioration. Voids or blisters were found in the waterproofing system. Delaminations could not be distinguished from blisters. Results of corrosion measurements were inconclusive. The waterproofing membrane prevented penetration of surface moisture. (FHWA)

349593 DA
STRENGTHENING OF THE FOUNDATIONS AND DECK OF THE BRIDGE OVER THE CHIUSELLA RIVER
Sansone, V
Permanent International Assoc of Road Congresses
Anas
Strade VOL. 83 NO. 1196 May 1981 pp 273-280 4 Fig. 1 Tab. 9 Phot. Italian
SUBFILE: TRRL; IRRD; HRIS
The bridge over the Chiusella River on the Siena-Betolle road, built in 1965, is a gerber structure of ordinary reinforced concrete, having 3 spans of 18, 24 and 18 M and a deck width of 16 M. This article describes and illustrates the methods and equipment used in the strengthening of the bridge and the repair of damage caused by rotation and settlement of the abutments and piers. Repairs included the renewal of bridge bearings, the replacement of bearing pads, and the waterproofing of expansion joints. (TRRL)

349333 DA
BRIDGE DECK SURFACINGS ON METAL FLOORS-2-FATIGUE BEHAVIOUR UNDER NEGATIVE MOMENT BENDING
Hameau, G; Puch, C; Ajour, AM
Laboratoire Central des Ponts et Chaussees
Bulletin de Liaison des Lab des Ponts et Chaussees N111 Jan 1981 pp 29-38 12 Fig. 4 Tab. 2 Phot. 1 Ref. French
SUBFILE: TRRL; IRRD; HRIS
The first surfacings laid on portable metal viaducts subjected to large repeated tensile stresses were very rapidly subject to cracking. To study this phenomenon, an apparatus simulating this negative mement bending was built to carry out fatigue tests on welded specimens. The comparison between the specimens and a real slab showed a good correlation between stress levels and stress localization. Two types of surfacing were tested: extra thin (thermo hardening binders spread with chippings, 5 to 10 mm thick) and thick (40 to 80 mm bituminous mixtures). (1) for the former: when the binder content is constant and sufficient (2 kg/sq.m), the ratio of aggregtes of the layers to the surface plays a dominant role in flexural fatigue strength; this type of surfacing, if the mix design and application are satisfactory, could be an acceptable solution for portable metal viaduct surfacings, strengthening being necessary during the 3 to 5 following years. (2) for the latter (thick surfacings): the nature of the waterproofing course is very important, the surfacing/support system appearing to behave like two girders linked by this surfacing; the thickness of the surfacing influences the process of deterioration: separation or cracking, caused by the different distribution of tensile stresses at the surfacing/steel interface and at the surface of the surfacing. (TRRL)

345775 DA
BRIDGE DECK DETERIORATION: TWO CASE STUDIES
Leslie, WG; Irwin, RJ; Chamberlin, WP
New York State Department of Transportation Engineering Research & Development Bur, 1220 Washington Ave Albany New York 12232 Res Rpt. 69; Federal Highway Administration 400 7th Street, SW Washington D.C. 20590
Feb 1979 Intrm Rpt. 43p
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA-NY-79-69;
CONTRACT NO.: HP&R
SUBFILE: HRIS
This report deals with surface condition of three concrete bridge decks constructed in 1966 and evaluated in 1973 and 1976. These were the first bridges in New York State for which complete performance and condition data were obtained on two separate occasions. Probably the study's most significant finding was the relationship between absorption of concrete over reinforcing steel and subsequent deterioration of the surface (for spans constructed with similar materials and having similar coverdepth distributions). Second, salt applications in this particular environment were estimated to be more severe than applications of salt solution on simulated decks in Federal Highway Administration studies. Conclusions are also drawn concerning the effect of concrete cover on performance, and on the effectiveness of a two-part bituminous-epoxy waterproofing membrane applied on one of the bridges. (FHWA)

341195 DA
EVALUATION OF SEVERAL BRIDGE DECK PROTECTIVE SYSTEMS
Weed, RM
New Jersey Department of Transportation 1035 Parkway Avenue Trenton New Jersey 08625; Federal Highway Administration 400 7th Street, SW Washington D.C. 20590
Aug 1980 Final Rpt. 86p
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA/NJ-81/003; 81-003-7783; PB81-233868
CONTRACT NO.: Study 7783; HP&R
SUBFILE: HRIS
This report details the installation and evaluation of ten different bridge deck protective systems. These include asbestos-modified asphalt, hot-applied liquid rubberized asphalt, two preformed sheet systems, one roll-on fabric system, latex-modified concrete, high density concrete, epoxy-coated reinforcing steel, galvanized reinforcing steel, and internally-sealed concrete. Most installations involve at least two test decks. Methods of evaluation included an appraisal by Construction, Maintenance, and Research engineers of the ease of installation of these systems. Apparent effectiveness has been judged by periodic visual monitoring and (for membrane systems only) the gathering of data from moisture sensing electrodes placed on the surface of the bridge deck. As described in the body of this report, the moisture sensing electrodes did not perform as effectively as expected, providing only very general trends rather than precise information. Also, because the quantity of detected moisture was relatively small, there is some uncertainty whether it represents moisture trapped in the pores of the concrete or water that has permeated the membrane after it was installed. Consequently, the evaluations are based on the impressions of the several engineers who witnessed these installations plus the periodic visual inspections by Research personnel. At this writing, the time in service of the various installations ranged from three to seven years. All are considered tentatively acceptable based on their performance to date although the Department's Design and Maintenance forces have developed distinct preferences based on ease of installation, cost and apparent effectiveness. All systems will continue to be monitored for several years as part of another study. As a further aid to designers, Appendix VI contains charts useful in making realistic comparisons between less expensive but shorter lived systems and more expensive but longer lived systems. For example, if the cost and expected life of a membrane system (short lived) are known as well as the cost of some more elaborate system, these charts enable the user to determine the expected life required for the more expensive system to be economically equivalent to the membrane system. (FHWA)

333632 PR
LONG TERM MONITORING OF BRIDGE DECK PROTECTIVE SYSTEMS
INVESTIGATORS: Croteau
PERFORMING ORG: New Jersey Department of Transportation 1035 Parkway Avenue Trenton New Jersey 08625
SPONSORING ORG: New Jersey Department of Transportation Materials Division; Federal Highway Administration Department of Transportation
CONTRACT NO.: 7799-12; HP&R
SUBFILE: HRIS
PROJECT START DATE: 8012
PROJECT TERMINATION DATE: 9009
The proposed project will monitor various bridge decks which have been installed with waterproof membranes and one each of the latex modified and high density concrete overlays. Evaluation will involve measurement of half-cell potential, resistivity and visual inspection.

331038 DA

DECISION CRITERIA FOR THE REHABILITATION OF CONCRETE BRIDGES DECKS

Manning, DG; Ryell, J

Transportation Research Board

Ontario Ministry of Transportation & Communic, Can

Transportation Research Record N762 1980 pp 1-9 4 Fig. 2 Tab. 33 Ref.

AVAILABLE FROM: Transportation Research Board Publications Office 2101 Constitution Avenue, NW Washington D.C. 20418

SUBFILE: HRIS

A systematic approach to bridge-deck rehabilitation is presented. Bridge-deck rehabilitation is consuming an increasing proportion of the resources of highway agencies. The nature and extent of deterioration are highly variable so that there is neither a single problem nor a single solution. The requirements for a condition survey are described. The performance of concrete overlays, water- proofing membranes, and cathodic protection applied to existing structures is assessed from field studies and the literature. Decision criteria that can be used to identify the most appropriate method of rehabilitation for any particular structure are given. (Authors) This paper appeared in Transportation Research Board Record No. 762, Corrosion, Cathodic Protection, Aggregate Upgrading, Concrete Density, and Pavement Markings.

319122 DA

CONCRETE-POLYMER MATERIALS FOR HIGHWAY APPLICATIONS

Fowler, DW

Texas University, Austin Center for Highway Research Austin Texas 78712 Res Rpt. 114-9F; Texas State Department of Highways & Public Transp Transportation Planning Division, P.O. Box 5051 Austin Texas 78763

Mar 1979 Final Rpt. 32p

AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161

REPORT NO.: FHWA-TX-79-03+114-9F; PB80-212012

CONTRACT NO.: Res Stdy 3-9-71-114; Contract

SUBFILE: HRIS

The use of concrete-polymer materials for highway applications has been studied with the objective of providing durable materials that would reduce maintenance in highway structures. Several significant developments resulted from this research. Partial-depth polymer impregnation was developed for improving durability of bridge decks. The process includes drying the concrete, cooling the surface, applying a low viscosity monomer solution to a sand cover, and permitting it to soak into the concrete, and applying heat to polymerize the monomer in the concrete to a depth of 0.5 in. (1.3 cm) or more. Post-tensioned polymer-impregnated beams were made and tested to determine the structural behavior. Beams with an I-shaped cross-section and a span of 8 ft (2.7 m) were fully impregnated with a monomer solution. High strength wire tendons were post-tensioned and the beams were tested to determine the flexural and shear behavior. Significant increases in strength and stiffness were observed. Time-dependent deflections were reduced by an order of magnitude. Polymer concrete was developed for repairing bridge decks. Clean, dry, graded aggregate is placed in the repair area and a monomer system that has sufficient promoter and initiator to polymerize at ambient conditions is applied to fully saturate the aggregate. Polymerization occurs in 30 to 45 minutes, producing a sound and durable repair. A polymer concrete overlay was developed to waterproof bridge decks. A thin layer of dry sand is covered with coarse aggregate. Two monomer solutions are applied and polymerization occurs at ambient temperature. (FHWA) Study conducted in cooperation with the Department of Transportation, Federal Highway Administration. Research Study Title: "Polymer-Impregnated Concrete for Highway Application".

309033 DA

WATER-PROOFING LAYERS IN CIVIL ENGINEERING CONSTRUCTION

Garcia, P

Buenos Aires Centro de Ingenieros

Revista de Ingenieria VOL. 26 NO. 100 Jan 1978 pp 25-30 2 Fig. 9 Ref. Spanish

SUBFILE: TRRL; IRRD; HRIS

This work tries to prove that at the present state of the art and world experience, the traditional asphalt layers are in the process of being substituted by new materials, such as resin-based layers, polymerised concretes, etc. and that no totally satisfactory solution exists in the water-proofing of civil engineering works. This is due to the fact that the working conditions of the coatings (wearing surface and water-proofing layer) in a bridge, for instance, are very different from those existing in the carriageway of a road, to the extent that new phenomena are seen to appear in certain types of structures, such as fatigue cracking. By way of suggestion, it would be extremely interesting to carry out full-scale tests and to this end build a branch road where it would be possible to allow traffic to run or be suspended at will in order to test the wearing surface and the water-proofing layers affording impermeability in carriageway joints, bridges, overpasses, etc. (TRRL)

300463 DA
PROTECTION EFFECTIVENESS OF THE WATERPROOF MEMBRANE SYSTEMS OF BRIDGE DECKS IN LOUISIANA
Ross, JE; Law, SM
Louisiana Dept of Transportation & Development P.O. Box 44245, Capitol Station Baton Rouge Louisiana 70804
Dec 1978 Final Rpt. 91 p.
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA-LA-78-121; PB-298980/4ST
CONTRACT NO.: Study No. 75-2C; HP&R
SUBFILE: HRIS; NTIS

The effects of de-icing salts on concrete and reinforcing steel is well documented, and efficient methods of protection from the penetrating chloride ions have long been sought. Concrete bridge decks need the protection, and waterproofing the concrete appears to be the answer. The aim of this research study was to evaluate the effectiveness of certain concrete bridge deck waterproofing membranes under conditions that prevail in Louisiana. This study included research evaluation of experimental installations on an exit ramp and a bridge on the Interstate highway in North Louisiana, as well as laboratory evaluations of the membrane systems. Laboratory evaluations included the tension test, bend test, heat-aged bend test and the electrical resistivity test. Field evaluations included visual observations and electrical resistivity tests. Installation procedures were monitored closely and a diary was kept of all activity. The primary conclusion reached on the study was that, using the electrical resistivity as the chief test for waterproofing ability, all the membrane systems evaluated in the field failed to achieve total impermeability of the concrete bridge deck. The primary recommendation of this research report is that the State of Louisiana not accept the concrete bridge deck waterproofing membrane systems herein for use as 100% waterproofing materials on concrete decks. /FHWA/ Conducted in cooperation with the Department of Transportation, Federal Highway Administration.

291136 DA
CONCRETE BRIDGE DECK WATERPROOFING SYSTEMS
Macdonald, MD
Building and Contract Journals Limited
Transport and Road Research Laboratory
Highways and Road Construction VOL. 41 NO. 1764 1973 pp 26-30 9 Phot. 8 Ref.
SUBFILE: TRRL; IRRD; HRIS

After briefly reviewing the types of deterioration in concrete bridge decks and their causes. The author outlines the requirements for an effective waterproofing system as set out in the revised technical memorandum no.BE27, and describes two tests developed at TRRL to appraise the waterproofing materials against the requirements listed in the memorandum. Waterproofing materials available in the United Kingdom are reviewed: mastic asphalt, extended epoxy and polyurethane resins, rubberized mastics, and prefabricated waterproof sheeting. Attention is drawn to the need for preparation of the concrete deck before applying the selected material. The problem of blistering is examined and preventive measures proposed. Details are given of the composition and application of protective layers to prevent damage to the membrane during construction work on site. Methods of preventing leakage through the waterproofing system beneath the curbing expansion joints, footpaths, central reservations and service ducts are mentioned together with the selection of suitable materials. /TRRL/

265240 DA
MATERIALS FOR WATERPROFFING CONCRETE BRIDGE DECKS
MacDonald, MD
Plastics Institute 11 Hobart Place London SW1 WOHL England
Transport and Road Research Laboratory
Conf Paper 6 pp
REPORT NO.: No. 3;
SUBFILE: TRRL; IRRD; HRIS

The paper describes the types of material currently available for waterproofing bridge decks, the conditions to which they are subjected on construction sites and methods for evaluating their suitability. Problems which occur when the materials are laid on bridge decks, such as that of blistering of the membrane, are then considered and various methods of preventing these difficulties are discussed. /Author/ Presented at the Conference Dampness in Building-Prevention Using Plastics.

265096 DA
BRIDGE WATERPROOFING-A VITAL FACTOR
Clarke, RD
British Tar Industry
Ruberoid Building Products Limited
Road Tar and Allied Binders VOL. 28 NO. 1 1974 pp 607 3 Phot.
SUBFILE: TRRL; IRRD; HRIS

The author highlights the main points from two reports on bridge waterproofing and suggests the use of hyload bridge waterproofing. The requirements of a waterproofing membrane, which will prolong the useful life of a bridge deck, are given. Materials for bridge waterproofing fall into 2 categories:- (1) Mastic asphalt, epoxy and polyurethane resins, and rubberised mastics which are applied in situ prefabricated sheetings of bitumen, polymers, vulcanished butyl or chloroprene rubber, propylene rubber and pvc. Hyload bridge waterproofing, which is a pitch polymer, falls into the second category and is available in two thicknesses, hyload 75 or 125. Its properties and laying procedures are given. One problem encountered in bridge waterproofing, included in the report by TRRL, is blistering. A blister occurs when the bond between the waterproofing membrane and the substrate is broken. This can be prevented by a pressure release layer between the deck and the membrane. /TRRL/

264329 DA
BRIDGE DECK WATERPROOFING
Frascoia, RI
Vermont Department of Highways
Public Works VOL. 105 NO. 6 .Jun 1974 pp 67-71 1974
SUBFILE: HRIS

The initial results are presented of field applications of various membrane systems installed on bridge decks to reduce freeze-thaw damage. Test results indicate that the preformed sheet systems (65-mil thick sheet composed of rubberized asphalt and non-tacky bituminous compound reinforced with a woven mesh and includes a prime coat and the use of a mastic along curb joints and at sheet termination points) show the most promise in preventing moisture and chloride solutions from entering the concrete. The major problem noted with such systems lies in satisfactorily sealing along curb lines and expansion dams. The application is being considered of a polyurethane coating along such areas. Laboratory tests are underway to determine the compatibility of the different products. In nearly all cases, liquid applied materials were plagued by pinholes, air bubbles or blisters due to outgassing of air or moisture vapor from the concrete and a number of different application techniques are reviewed which may be used to reduce their number. A laboratory study has been initiated to obtain information on the rate of chloride penetration through membrane coatings containing specific numbers of pinholes or bubbles.

264790 DA
POLYMER-IMPREGNATED CONCRETE SURFACE TREATMENTS FOR HIGHWAY BRIDGE DECKS
Fowler, DW; Houston, JT; Paul, DR
American Concrete Institute, Journal of VOL. 70 NO. 11 .Nov 1973 Proceeding p 765 1973
SUBFILE: HRIS

The significant increases in strength and durability of polymer-impregnated concrete sugges the use of this material as a surface treatment for highway bridge decks. Several monomers have been used successfully o obtain treatments that can be field applied. The concret slab surface is kept wetted with the monomer for severa hours. External heat is supplied by one of several methods to polymerize the monomer to a depth of 0.5 to 1.5 in. (1.37 to 3.8 cm). Evaluation tests on the treated specimens indica excellent resistance to freeze-thaw deterioration, water penetration, abrasion, and wear. Skid resistance has generally been found to be equal to or superior to nontreated slabs. Th treatments appear to be practical and economically feas ble f application to highway bridge decks. /ACI/ These abstracts re brief summaries of all ACI technical material publis d outside of these proceedings and are indexed in the Annua Proceedings Index. Number SP 40-5.

261546 DA
EVALUATION OF A BRIDGE DECK WATERPROOFING SYSTEM
Fera, JD; Fruggiero, RL
Rhode Island Department of Transportation; .Materials Section; Providence; Rhode Island; 02903
Jul 1974 74 pp Figs. 4 Tab. 8 Ref. 1974
SUBFILE: HRIS
Nine Rhode Island bridges, varying in age from 0 to 10 years and overlayed with 3 inches of asphalt pavement over a 3-ply glass fabric and asphalt waterproofing membrane system, were tested for membrane porosity and reinforcing steel corrosion. Based upon the test criteria that membrane resistance readings of less than 100,000 Ohms/sq ft indicate a poor waterproofing seal, the waterproofing system was found to become over 90% ineffective within 3 years of bridge use, with the membrane-curb and membrane-expansion plate seals suspect upon installation. Three mechanisms of membrane breakdown are assumed and discussed in light of the data obtained in the survey. Based upon the criteria that half-cell potentials numerically greater than -0.30 Volts indicate the presence of active corrosion in the reinforcing steel, corrosion in the bridge decks surveyed was found minimal (0-5% of total deck area) and concentrated exclusive at deck joint and curb locations. Although the most extensive areas of corrosion were found along exposed expansion plates, active potentials were also detected in the vicinity of overlayed joints, primarily at the joint-curb junction. A tentative lower limit for the time to corrosion of the reinforcing steel is estimated.

260705 DA
NEW APPROACHES TO BRIDGE DESIGN
Leyden Publishing Company, Limited
Building Research and Practice VOL. 1 Jan 1973 pp 37-41 6 Fig.
SUBFILE: TRRL; IRRD; HRIS
The article describes recent advances in bridge design in Germany. The contact connection of precast prestressed units has enabled standardization to be made of precast units in a range of sizes, enabling the ordering of 'packaged' bridges. The force-locking contact connection eliminates in-situ concreting and also allows the design of demountable structures. The Brudermuhl bridge in Munich, erected in eight days to relieve traffic congestion, is described. The 190M long bridge used units of up to 30M in length, having stepped ends which provided the bearings. In the absence of in-situ concreting, the tendons are protected with epoxy resin in place of the usual grout. The structure is supported on square precast columns placed in socket foundations cast in the ground. Waterproofing is with mastic asphalt overlaid by a bituminous wearing carpet. A patent carriageway expansion joint consisting of a hollow neoprene filter gripped between epoxy nosings is described in detail. The other major development is the prestressed concrete cantilever. This system was used in a bridge over the river main connecting two halves of a factory in Frankfurt. Because of site and navigation constraints, the construction depth was limited to 2.6M in a structure having a clear span of 148m, and limitations on the length of the landward portion meant that a cable-stayed girder could not be long enough to complete the river span. The gap was made up by a balanced pre-stressed cantilever from the opposite bank. /TRRL/

218962 DA
BAY STATE CONSIDERS ALTERNATIVE BRIDGE DECK REPAIRS
Hagenbuch, JJ
Public Works Oct 1971
SUBFILE: HRIS
since there are never enough funds for all the maintenance activities, preventative action is often delayed. The standard practice of overlaying all newly constructed concrete bridge decks with A membrane water proofing and a riding surface of bituminous concrete is described. The three membrane systems currently under evaluation are: (1) a liquid membrane wateproofing consisting of a hot rubberized asphalt compound applied by squeezee to a primed concrete deck surface, producing a film thickness of approximately 150 miles. (2) an elastomeric waterproofing barrier consisting of uncured synthetic rubber sheets laid directly on a concrete deck primed with rc-70. The sheets are about 100 mills thick and (3) a bridge deck paving system consisting of a primer made up of a blend of tar-oil, rubber and solvent followed by a hot-applied adhesive of blended rubber and tar. This is covered with sheets of a plastic membrane. The question - on an exposed concrete deck that begins to disintegrate and which has been exposed to heavy chloride application for several years, will overlay that is a good barrier also prevent further disintegration of the underlying concrete? - is discussed. /author/

218592 DA
BRIDGE DECK PROTECTION AND RESURFACING
Pettitt, RA
ASCE Journal of the Structural Division Apr 1970 Vol 96, No ST 4, PROC PAPER 7217 PP 791-802
SUBFILE: HRIS
AS A RESULT OF THE OHIO RIVER BRIDGE FAILURE AT POINT PLEASANT, WEST VIRGINIA IN 1967, PERIODIC INSPECTIONS OF BRIDGES ARE REQUESTED BY THE U.S. BUREAU OF PUBLIC ROADS. THE KEY TO EXTENDED LIFE OF BRIDGE DECKS, AND SOMETIMES TO THE SAFETY OF THE ENTIRE BRIDGE STRUCTURE, IS FOUND IN MAINTAINING THE WATER-TIGHT INTEGRITY OF THE DECK. PERIODIC INSPECTIONS SHOULD LOGICALLY LEAD TO TIMELY PREVENTIVE MAINTENANCE PRACTICES. THE HISTORY OF THE LOS ALAMOS CANYON BRIDGE AT LOS ALAMOS, NEW MEXICO, AND THE CONTINUING INVESTIGATION OF PROTECTION MATERIALS IS USED AS A TYPICAL EXAMPLE OF BRIDGE DECK INSPECTION AND MAINTENANCE PRACTICES. PROTECTION METHODS ARE DIVIDED INTO TWO CATEGORIES: PROTECTIVE COATINGS FOR WATER PROOFING THE ORIGINAL BRIDGE DECK SURFACE; AND PROTECTIVE COVERINGS WHICH PROVIDE A NEW WEARING SURFACE AS WELL AS SEALING AND WATERPROOFING THE DECK. /AUTHOR/

218438 DA
PROTECTION FOR CONCRETE BRIDGE DECKS BY MEMBRANE WATERPROOFING
Hagenbuch, JJ
Highway Research Record, Hwy Res Board 1968 No 254, pp 58-60
SUBFILE: HRIS
standard practice in Massachusetts is to use a membrane waterproofing on all new construction of concrete bridge decks that are to be covered by bituminous concrete. Restoration of concrete bridge decks that have deteriorated to the point of requiring repairs, include membrane waterproofing as a part of the restoration project. Bridges built prior to 1940 without adequate deck waterproofing that have a bituminous concrete riding surface of various thicknesses are presently being restored. The 1958 department specifications call for membrane waterproofing and for 2-ply coated woven-glass fabric placed between 5 moppings of a coal tar pitch emulsion. Procedures and costs are reviewed. The emulsion is thoroughly agitated in its container and no adulterants of any nature were added. Where membrane waterproofing is placed at steel expansion joints, scuppers, manholes or other metal projecting through the concrete, the membrane is turned up about 1 1/2 inches and sealed to the metal. Bleeders, 2 inches in diameter, are installed at the curb lines, prior to the placing of the membrane waterproofing. The waterproofing is protected throughout all operations of placing the bituminous concrete base course. No vehicles are permitted on the bare waterproofing. The first course of bituminous concrete must be spread by hand. Performance observations show satisfactory results from this method of waterproofing. No other method or materials have been found that will produce comparable results at such a reasonable cost.

218147 DA
DEVELOPMENT OF A BRIDGE DECK PROTECTIVE SYSTEM
Riley, O
Highway Research Record, Hwy Res Board 1967 No 173, pp 13-24, 11 FIG, 2 TAB, 16 REF
SUBFILE: HRIS
In the development of a bridge deck protection system, waterproofing is the key to prevention and coal-tar epoxy resin is the most suitable membrane because of its history of satisfactory service, its strength and flexibility, ease of application and cost. The best wearing course is a dense asphaltic concrete /for impermeability/ fortified with asbestos /for stability/ and modified with latex /for flexibility/. Such a wearing course doubles the cost of the system but increases its service life at least five-fold. The total cost of a protective system is less than half the total cost of repair. Other factors such as design procedures, methods of construction, and skid-resistance are unimpaired.

217357 DA
WATERPROOF MEMBRANES FOR PROTECTION OF CONCRETE BRIDGE DECKS
Novak, JR
American Assn of State Hwy & Transp Office Proc Dec 1971 12 pp 1 Fig 3 Tab 1 Ref 1971
SUBFILE: HRIS
The development of effective waterproofing membrane systems for use on concrete bridge decks in reported. This paper reports the results of the first phase of the study. A comprehensive literature search revealed 380 documents that bear on bridge deck protection. All information available on membranes was analyzed, and membrane service requirements and design criteria were defined. A field inspection program was conducted to evaluate selected systems, including application techniques. The field data acquired is tabulated. Information from the field study and literature survey was used to identify membranes and materials that have consisted performance in the field. Material characterization tests (laboratory) are reported which identify and define those properties that affect performance of membrane systems. Qualifying tests (relative to field performance) were devised. The laboratory performance screening test program is tabulated. An experimental program evaluating the performance in the field of selected membrane systems under service conditions was devised, as was also, a procedurd for determining cost-benefit ratios associated with the use of membrane systems. Tentative conclusions based on the study are presented.

217248 DA
AN ELECTRICAL METHOD FOR EVALUATING BRIDGE DECK COATINGS
California Division Highways; .Spellman, DL; Stratfull, RF; Trimble, RR; Dukelow, S; Halterman, J
Jun 1971 p 28 1971
SUBFILE: NTIS; HRIS
field and laboratory tests have indicated that the electrical resistance of a bridge deck coating can be related to the voids and thus sealing ability of the coating. It is considered that this nondestructive method for evaluating bridge deck coatings may be an additional tool for evaluating the performance of membranes used to prevent the ingress of deicing salts which cause corrosion of the steel. In this method, the resistance between various locations on the bridge surface and the reinforcing steel is measured. It is postulated that an excellent waterproof coating for bridges would always have a resistance greater than 500,000 ohms/square foot, while a poor or perforated coating would never have a resistance greater than 100,000 ohms/square foot. /author/

216536 DA
THE USE OF SYNTHETIC RESINS IN ROAD CONSTRUCTION, INDUSTRIAL FLOOR COVERING, AND WATERPROOFING /IN FRENCH/
Ansart, R
Annales de l'Institut Tech du Batiment Travaux Pub Nov 1968 No 251, pp 639-666, 22 FIG, 2 TAB
SUBFILE: TRRL; IRRD; HRIS
the use of polyvinyl chloride, epoxy and polyurethane resins, with the possible addition of tar, in the manufacture of bituminous road binders, industrial floor coverings and waterproofing products are discussed. Polyvinyl chloride added to coal tar improves the execution of road surfacings, and is used for executing waterproof joints. Epoxy resins can be utilized for protecting concrete surfaces; added to tar, they are increasingly used in the construction of waterproofing courses on bridge decks, in high performance wearing courses, and in the preparation of smoothing mortars, polyurethane resins can be used for protecting metals surfaces (orothropic slabs) against corrosion and wear, and for scaling concrete paving joints. /lcpc(a)/rrl/

216013 DA
BRIDGE DECK MEMBRANES, EVALUATION AND USE IN CALIFORNIA
Spellman, DL; Stratfull, RF
California Department Transportation; ./Transportation Laboratory
Nov 1973 Interim 11 pp Contract D-3-11 1973
SUBFILE: HRIS
Laboratory tests used to screen bridge deck membranes are (1) tension on the crack bridging; (2) puncture ability to withstand paving operations, and (3) bending or cold temperature ductility with and without heat aging. None of the tests, however, are known to be related to the service life of the membrane. Results of comparing field and laboratory tests are given. It was found that a 240 deg F asphalt concrete pavement could cause a rise in temperature of the membrane to about 170 deg F for approximately 3 minutes. However, windrowing of the pavement was found to melt some membranes due to the long term sustaining of high temperature. Tests were made using the electrical resistance method as an indicator of impermeability of A membrane. It was found that the electrical resistance will vary with time. The mathematical relationship between resistance and time is given for 8 field tests. /FHWA/

213798 DA
CONCRETE BRIDGE DECK DURABILITY
Hrb Nchrp Synthesis of Hwy Practice 1970 No 4, Nchrp PROJECT 20-5, 28 PP, 13 FIG, 3 TAB, 102 REF
SUBFILE: HRIS
A THOROUGH LITERATURE SEARCH WAS MADE OF ALL PERTINENT PUBLICATIONS, INTERVIEWS WERE HELD WITH KNOWLEDGEABLE HIGHWAY PERSONNEL, AND CURRENT PRACTICES, PLANS, SPECIFI- CATIONS MANUALS. AND RESEARCH RECOMMENDATIONS WERE EXAMINED TO ACQUIRE ALL DATA PERTAINING TO THE PROBLEM OF DETERIORATING CONCRETE BRIDGE DECKS, THE CAUSES, PREVENTION, AND CORRECTIVE MEASURES. THERE APPEARS TO BE A SIMILARITY OF BRIDGE DECK DETERIORATION THROUGHOUT THE UNITED STATES FALLING INTO THE CATEGORIES OF THREE TYPES OF DEFECTS: SPALLING, SCALING, AND CRACKING, SPALLING IS GENERALLY RECOGNIZED AS THE MOST TROUBLESOME DEFECT, BECAUSE THE DECK IS WEAKENED LOCALLY, REINFORCEMENT IS EXPOSED, RIDING QUALITY IS IMPAIRED, AND REPAIR WORK IS DIFFICULT. SPALLING IS MAINLY CAUSED BY THE CORROSION OF REINFORCING STEEL, REQUIRING THE PRESENCE OF MOISTURE AND A CHLORIDE SALT. CRACKS PROVIDE READY ACCESS FOR MOISTURE AND SALT TO REACH THE STEEL, ALTHOUGH COURSE CONCRETE WITHOUT CRACKS IS ALSO SUSCEPTIBLE TO MOISTURE AND SALT INTRUSION. VARIOUS WATERPROOF BARRIERS PROTECTED BY A WEARING COURSE ARE CURRENTLY IN VOGUE AS A PREVENTIVE MEASURE. IN ADDITION, GREATER COVER OVER REINFORCING STEEL, INCREASED EFFORTS AT CRACK CONTROL, AND LESS POROUS CONCRETE ARE URGED AS ESSENTIAL IMPROVEMETNS. SCALING CAN BE VIRTUALLY ELIMINATED BY THE USE OF HIGH-QUALITY AIR-ENTRAINED CONCRETE, ASSISTED WHEN NECESSARY BY PERIODIC LINSEED OIL APPLICATIONS. CRACKING IS NOT CONSIDERED TO BE SERIOUS.

211288 DA

REPAIR OF THE SURFACING OF A BRIDGE DECK BY GUSSASPHALT

Cartallas, E; Phelep, M; Benoit, O

Bull Liaison Labs Routiers /France/ N53 Jun 1971 pp 85-96 8 Fig 5 Tab 11 Phot 3 Ref

SUBFILE: TRRL; IRRD; HRIS; IRF; IRRD

THE WATERPROOFING COURSE CONSISTS OF ASPHALT POWDER AND BITUMEN WITH A POLYPROPYLENE ADDITIVE TO IMPROVE THE MECHANICAL PROPERTIES OF THE MATERIAL. THE COURSE WAS APPLIED IN THE CONVENTIONAL WAY. THE BASE COURSE HAS A STANDARD GRANULOMETRIC COMPOSITION, AND CONTAINS A HARD BITUMEN MADE THIXOTROPIC BY THE ADDITION OF A LIMESTONE FILLER AND POLYPROPYLENE. THE THIXOTROPIC PROPERTIES OF THE BINDER ARE MAINLY USED IN THE WEARING COURSE; DURING SPREADING, THE AGGREGATES ARE RETAINED BY MEANS OF A MOBILE, VARIABLE-SPEED RAKE FIXED ON THE FINISHER'S BLADE, THUS GIVING NON-SKID CHARACTERISTICS SUPERIOR TO THOSE OF STANDARD GUSSASPHALT. /IRRD-IRF/

210234 DA

WATERPROOFING OF CONCRETE BRIDGE DECKS

Oecd, Paris /France/

Jul 1972 R&d Rept 89 pp 11 Fig 3 Tab 2 Phot 29 Ref

SUBFILE: TRRL; IRRD; HRIS

THE REPORT DISCUSSES THE REQUIREMENTS FOR AN EFFECTIVE PAVEMENT WATERPROOFING LAYER, DESCRIBES THE SYSTEMS AVAILABLE (BITUMINOUS MATERIALS, PREFABRICATED SHEETING AND THIN ADHESIVE MEMBRANCES), REVIEWS THEIR USE IN VARIOUS COUNTRIES, AND EVALUATES CONCRETE SURFACINGS WITHOUT SPECIAL WATERPROOFING. IT PRESENTS A CRITICAL APPRAISAL OF THESE SYSTEMS TOGETHER WITH CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH. THE APPENDICES CONTAIN DETAILS OF TESTS AND SPECIFICATIONS FOR WATERPROOFING MATERIALS. /TRRL/

209812 DA

WATERPROOFING OF CONCRETE BRIDGE DECKS

Organ Econ Co-operation & Devel; ./Road Research Group

Jul 1972 89 pp 1972

SUBFILE: HRIS

In some countries an unexpectedly large amount of deterioration has been experienced in the concrete decks of bridges built during the past two decades. Experience has shown that under heavy traffic, full exposure to weather, and in the corrosive environment produced by the regular application of de-icing salt in winter, the structural concrete needs the protection given by an effective waterproofing layer. The report discusses the requirements for an effective waterproofing layer, describes the systems available (bituminous materials, prefabricated sheeting, and thin adhesive membranes), reviews their use in various countries, and evaluates concrete surfacings without special waterproofing. It then makes a critical appraisal of these systems and ends with conclusions and recommendations for further research. The appendices contain details of tests and specifications for waterproofing materials. /author/

209808 DA
BRIDGE DECK LEAKAGE IN MAINE-INTERIM REPORT
Maine Dept of Transportation; .Smith, EV
Feb 1972 62 pp 6 Fig 1 Tab 6 Phot 35 Ref 1972
SUBFILE: HRIS
the effectiveness of latex concrete in preventing bridge deck deterioration is based on results of 13 years service. Its value in extended life should warrant wider use, especially for thin overlays. Its present competitors, expanded shale or metallic aggregates as used in New York and New Jersey, are probably more suitable only where materials are nearby and expertise is available in all procedures. The Maine system of waterproofing membrane using type C (1968 spec.) or type C (1960 supp.) asphalt has been in service about 10 years on many I-95 bridges. Considerable leakage has been found on most of the continuous spans and seepage comes directly through cracks in the membrane. On simple spans leakage is not as severe but surface water causes problems at the expansion joints. Field photos and cores plus lab tested samples from five membranes, six to eight years old on I-95 and I-395 in the bangor area, show that the cracks in the pavement often extend through both membrane and pavement and the membrane becomes brittle with age and/or overheating during placement. Other sources of membrane leakage are the longitudinal and transverse joints in slabs and curbs. The pavement at the granite curb contracts in cold weather and the mortar does not bond well to bottom of granite causing seepage and motar deterioration. Also, leakage at cu b contraction joints allows calcium and chlorides to eat away paint from the underside of beam or girder top flanges. Corrosion fatigue of tension flanges can develop in regular steels which are brittle at low temperatures and also threaten the corrosion resistant high strength steels used in girders today. The older concrete bridges built in the twenties and thirties are shown to be more resistant to salt damage due to the longer curing period before exposure to salt. The newly developed California electrical potential method provides excellent means of monitoring bridge decks for salt corrosion damage. It was used in Maine in 1971 on a reconstructed I-95 bridge containing no membrane and on many bridges with test membranes. Other methods of waterproofing have been tried in Maine. These include coal tar emulsion membranes, epoxy overlays, epoxy surface sealants, epoxy sand slurries, asbestos base and wearing course, bituminous (cationic) slurry seals and rubberized membrane. Bonded concrete wearing courses, monolithic concrete wearing courses, linseed oil treatment and non-shrink grout have been put into regualr service use. Hot rubberized asphalt, polyurethanes and polysulfides are used as joint sealants. Open joints are being closed with preformed seals and elastomeric and modular devices which are still under evaluation. /author/

209611 DA
WATERPROOFING RAILROAD BRIDGE DECKS
Drew, FP
American Railway Bridge & Building Assoc, Proc pp 63-70, 3
FIG, 6 PHOT
SUBFILE: HRIS
the standard bituminous membrane system of waterproofing is compared with using butyl rubber sheeting. Whereas the bituminous membrane requires may lap splices, the butyl rubber sheeting can be unrolled in one continuous sheet to cover the entire bridge deck. Also, butyl as a synthetic has other advantages: (1) it remains flexible and elastic over a wide temperature range; (2) it is impermeable to water; (3) it resists tearing and abrasion; (4) it resists aging by sun, weather, and temperature; (5) it is easy to apply, since no special equipment is required; and (6) it costs less per square foot.

205282 DA
WATERPROOF MEMBRANES FOR PROTECTION OF CONCRETE BRIDGE DECKS
NCHRP Research Results Digest N56 Mar 1974 13 pp 1974
AVAILABLE FROM: Transportation Research Board Publications Office 2101 Constitution Avenue, NW Washington D.C. 20418
SUBFILE: HRIS
A project is reported, which was undertaken to discover improved waterproof membrane protective systems. A hundred and forty seven known membrane systems were subjected preliminary examination which included inspection of the performance of 25 systems in place on existing bridge decks. Seventy eight systems were selected for detailed study and 9 were selected for a field application test. Laboratory tests were performed to determine permeability, crack-bridging capability, durability resistance to impact damage, and structural serviceability. Fire systems consisting of vulcanized, cured, or cross-linked elastomers, were selected for further evaluation. All five systems provide good dimensional stability on exposure to asphaltic concrete at application temperature, to water, to solar heat, and to freeze-thaw conditions. All these systems require a protective coat of asphaltic concrete to serve adequately, and all but one require application of an intermediate protective layer to avoid damage during construction operations following membrane installation. Tentative specifications developed for these systems are presented. Recommendations are made for field evaluation tests relating to electrical resistance and tensile bond. The equipment and materials used in the tests are described and the procedures are outlined.

192955 PR
EVALUATION OF WATER PROOFING SYSTEMS IN PREVENTING BRIDGE DECK DETERIORATION
INVESTIGATORS: Strong, MP
PERFORMING ORG: North Carolina Department of Transportation Division of Highways P.O. Box 25201 Raleigh North Carolina 27611
SUBFILE: HRIS
PROJECT START DATE: 7501
PROJECT TERMINATION DATE: 8407
The objective of this study is to evaluate the effectiveness and relative cost of sealing bridge decks against moisture and salt intrusion by the use of water proofing membrane systems applied to four separate bridge decks. Electrical resistance test will be taken to determine when the membrane systems have been broached by sodium chloride. Electrical potential tests will then be taken to determine when significant reinforcing steel corrosion has occurred.

192836 PR
EVALUATION OF PROMISING PROPRIETARY BRIDGE DECK EXPANSION JOINT DEVICES
INVESTIGATORS: Bashore, FJ
PERFORMING ORG: Michigan Department of Transportation Research Laboratory Section P.O. Box 30049 Lansing Michigan 48909
SPONSORING ORG: Michigan Department of Transportation
SUBFILE: HRIS
PROJECT START DATE: 7801
PROJECT TERMINATION DATE: 8209
The performance of various selected FHWA approved waterproof expansion dams is to be closely followed through systematic field inspections. The installation of these dams, of the continuous strip seal design, is to be observed to note any problems or factors peculiar to the selected dam which may affect the long term performance under heavy traffic and temperature movements. The long term performance of the various proprietary dams is to be evaluated as per FHWA Notice 5140.11, dated 10-25-77.

188505 DA
A STUDY OF DETERIORATION IN CONCRETE BRIDGE DECKS
Missouri State Highway Department; .Division of Materials and Research, State Highway Building; Jefferson City; Missouri ; 65101
Apr 1978 Final Rpt. 90 p. 1978
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA-MO-62-1; PB-291937/AS
CONTRACT NO.: Study No. 62-1; Contract
SUBFILE: HRIS; NTIS
Seven phases of study related to the problem of bridge deck deterioration are reported. The variables include timing variations for completion of hauling and finishing deck concrete, concrete test variations, and weather variations; an increase in depth of cover over the top reinforcing steel from 1 1/2 to 2 inches; insulating the underside of a bridge deck; sealing different areas of a bridge deck with one and five applications of linseed oil and with one application of a penetrating epoxy sealer; using Dow Corning 777 admixture in bridge deck concrete; evaluation of various field applied waterproofing memberanes for effectiveness in protecting bridge decks (some of these membranes are no longer available); and determining chloride content of concrete from bridge decks with asphaltic concrete overlays with and without protective membrane systems. /FHWA/ Sponsored by the Missouri State Highway Department. Study was conducted in cooperation with the Department of Transportation, Federal Highway Administration.

185255 DA
EVALUATION OF BRIDGE DECK REPAIR AND PROTECTIVE SYSTEMS
Swanson, HN; Donnelly, DE
Colorado Department of Highways; .Planning and Research Branch, 4201 East Arkansas Avenue; Denver; Colorado; 80222
Dec 1977 Intrm Rpt. 33 p. 1977
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA-CO-78-77-8; PB-289292/5ST
SUBFILE: HRIS
This report describes efforts to control bridge deck deterioration in Colorado. A follow up testing program to determine the degree of continued effectiveness of waterproofing membranes has been underway since 1974. Results to date show that the most important factor is how well a membrane is placed. Several projects in Colorado have employed the use of Latex Modified Concrete as a surface wearing and protective course. Recent inspections of latex modified bridge decks indicate that some significant cracking is beginning to take place. Chlorides are entering these cracks and steel corrosion may be in progress at these points. Considerable time may be required, however, to determine if the corrosion progresses laterally and the true value of this design. /FHWA/ Sponsored by Colorado State Department of Highways and prepared in cooperation with the Federal Highway Administration.

182676 DA
WATERPROOFING OF ENGINEERING STRUCTURES
Gordillo, J
Portavoz de la Agrupacion Nacional de Fabricantes
Alemas N93 Apr 1976 pp 23-27 6 Fig. 2 Tab. 5 Phot. 14 Ref. Spanish
SUBFILE: TRRL; IRRD; HRIS
Increased interest has been taken in recent years in the problem of the impermeability of bridges. The problem arises from the development of the road network and from more widespread use of lightweight structures of prestressed or post-tensioned concrete, where protection against water is necessary to ensure good maintenance of the structure. There are three reasons why it is necessary to make the decks of bridges impermeable: (1) the problem of the strength of the material, (2) the right material which constitutes the deck, (3) external agents. An impermeable course should present the following characteristics: (A) maximum adaptability to surface irregularities, (B) high tensile strength, (C) maintenance of visco-elastic properties at low temperatures, (D) possibility of applying a layer of desired thickness with a suitable technique, (E) resistance to ageing. Current methods of achieving impermeability are analyzed, stressing the most important: (1) asphalt membrane, (2) systems of bitumen-rubber, (3) systems with a pitch-epoxy base. The advantages and disadvantages of each system are analyzed. /TRRL/

182637 DA
BRIDGE DECK REHABILITATION: PART II OKLAHOMA'S EXPERIENCE WITH BRIGE DECK PROTECTIVE SYSTEMS
Ward, PM
Oklahoma Department of Transportation; .200 NE 21st Street; Oklahoma City; Oklahoma; 73105
Jun 1978 151 pp 1978
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA-OK-78-71-04-1; PB-285281/2ST
CONTRACT NO.: DOT-FHWA-71-04-1; Contract
SUBFILE: HRIS; NTIS
The objectives of this project were to study the factors involved in bridge deck deterioration and to formulate and evaluate techniques to prevent or halt such deterioration. Part II of this project was concerned with the evaluation of two types of bridge deck protective systems; waterproof memberanes and latex modified Portland Cem'nt concrete overlays. This report contains construction and electrical resistance data on nine different memberane systems applied to 35 structures. Also included is construction data on latex modified overlays applied to 5 structures. One of these structures is six years old and the results of the yearly bridge surveys are included. /FHWA/ This report was sponsored by the Federal Highway Administration.

177669 PR
THE PROTECTION EFFECTIVENESS OF WATERPROOF MEMBRANE SYSTEMS ON BRIDGE DECKS IN LOUISIANA
INVESTIGATORS: Ross, JE; Law, SM
PERFORMING ORG: Louisiana Dept of Transportation & Development P.O. Box 44245, Capitol Station Baton Rouge Louisiana 70804
SPONSORING ORG: Louisiana Dept of Transportation & Development; Federal Highway Administration Materials Division
CONTRACT NO.: 75-2C; HP&R
SUBFILE: HRIS
PROJECT START DATE: 7504
PROJECT TERMINATION DATE: 8006
The aim of this fundamental research is to study and ascertain the effectiveness of bridge deck waterproofing membranes under conditions that prevail in Louisiana. This study will evaluate the experimental installation by Department maintenance forces of five types of membranes on the Bayou Lafourche Bridge and on an access ramp to the Outachita River Bridge (both facilities on Route I-20). This research effort will include both laboratory and field evaluation. Of primary concern will be behavior of the membrane-asphaltic concrete overlay system when subjected to the extreme heat of the summer months and the freeze cycles of northern Louisiana's winter months.

176804 DA
ANALYSIS OF PROBLEMS IN PROTECTIVE COATINGS APPLICATION
Montle, JF
Carboline Company
Corrosion 11 pp 5 Ref. 1977
AVAILABLE FROM: Engineering Societies Library 345 East 47th Street New York New York 10017
REPORT NO.: Paper 2;
SUBFILE: EIT; HRIS
This paper discusses some actual problems. The examples given are: case of the disappearing zinc -- This particular problem involved the coating of steel on the substructure of a series of highway bridges; The tiger stripe phenomena -- It involved the use of an inorganic zinc primer as a lining for the suppression chamber of a boiling water nuclear power generating facility; the case of the spongy membrane -- This problem involved the application of a rapid curing elastomeric polyurethane membrane, specifically developed, for use as a waterproofing membrane on highway bridge decks; the case of the mysterious crystals -- This problem was manifested as excessive tip clogging during airless spraying of high build epoxy polyamide coating; and the case of the lining failure -- This involved the application of a modified phenolic, tank lining coating to a multi-stage flash evaporation distillation unit. This paper was presented at the International Corrosion Forum San Francisco, California, March 14-18, 1977.

174105 DA
INVESTIGATION INTO THE ORIGIN OF BLISTERING OF--WATERPROOFING COURSES ON CONCRETE BRIDGE DECKS
Beijers, GMH
Arkitektens Forlag
Technische Hogeschool Delft, Netherlands
Asfalt N4 Dec 1976 pp 122-127 5 Phot. Dutch
SUBFILE: TRRL; IRRD; HRIS
The phenomenon of blistering is described. Two types of blistering may be differentiated: (1) blistering during, and (2) blistering after the construction of the waterproofing course. The theoretical aspects of these two types of blistering are discussed. Laboratory tests of the second type of blistering were carried out: courses of various bituminous mixtures were laid on wet as well as on dry concrete slabs and subjected to heat radiation. It appeared that the presence of moisture or water vapour is not necessary for the development of blisters, whereas imperfect adhesion between the several layers is an important factor.

172505 DA
DEVELOPMENT OF A COLD-POURED BRIDGE DECK MEMBRANE SYSTEM
Meader, AL, Jr; Schmitz, CG; Henry, JE
Chevron Research Company
ASTM Special Technical Publications N629 pp 164-177 8 Ref. 1977
AVAILABLE FROM: Engineering Societies Library 345 East 47th Street New York New York 10017
SUBFILE: EIT; HRIS
A two-component asphalt-extended urethane membrane was developed which satisfies the requirements demanded of a waterproofing membrane for protecting reinforcing steel in concrete bridge decks from corrosion. A technique for application of the membrane is described which essentially eliminates blister formation. Presented at the Chloride Corrosion of Steel in Concrete Symposium, 79th ASTM Annual Meeting, Chicago, Illinois, June 27-July 2, 1976.

165779 DA
WATERPROOFING MEMBRANES FOR BRIDGE DECK REHABILITATION
Chamberlin, WP; Irwin, RJ; Amsler, DE
New York State Department of Transportation; .Engineering Research & Development Bur, 1220 Washington Ave; Albany; New York; 12232
May 1977 Final Rpt. 48 pp 1977
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: FHWA-NY-77-59-1;
CONTRACT NO.: FCP 44B1-164; HPR
SUBFILE: HRIS
Measurements of electrical potential and resistance on 44 rehabilitated reinforced concrete bridge decks in New York State are reported. Twenty included a two-component, liquid-applied bituminous-epoxy waterproofing membrane--New York's standard from 1958 through 1974. Ten others included a modified polyester-resin, and 14 had a variety of experimental membranes. While the bituminous-epoxy membrane system failed to perform as hoped, it has probably been no more nor less effective than membranes in general use before 1970 in other parts of the United States. Other membrane types used experimentally in New York have performed better, as a group, than the bituminous-epoxy, but not outstandingly so. None of the membranes evaluated was successful in preventing continuation of corrosion of steel reinforcement where chlorides were already present in the decks. The degree of roughness of the concrete surface on which the membranes were placed appeared to be an important factor determining their successful performance. /FHWA/ Sponsored by DOT, Federal Highway Administration.

163029 DA
DURABILITY OF STEEL-FORMED AND SEALED BRIDGE DECKS
Cady, PD; Renton, JB
Pennsylvania State University, University Park; New York State Department of Transportation
Transportation Research Record N613 pp 14-20 7 Fig. 1 Tab. 7 Ref. 1976
AVAILABLE FROM: Transportation Research Board Publications Office 2101 Constitution Avenue, NW Washington D.C. 20418
SUBFILE: HRIS
A 2-year research project was carried out to investigate the freeze-thaw durability of concrete bridge decks cast on steel forms, which remain in place after construction, and sealed on the top surface with a waterproof membrane. The long-term durability of the forms was also studied. Laboratory freeze-thaw tests equivalent to a winter season and one winter of outdoor exposure tests were carried out on simulated bridge-deck slabs. These slabs covered all combinations of the following variables: (a) form type (wood and steel) and (b) surface treatment (non, linseed oil, and waterproof membrane). In addition, 25 bridge decks with steel forms and waterproof membranes and 1 bridge deck without steel forms but with a membrane were inspected. The bridge decks ranged in age from 1 to 13 years, and all but six were 8 or more years old. By using a variety of inspection techniques that ranged from visual examination to pluse velocity measurement, it was determined that steel-formed bridge decks with surface sealing are no more prone to freeze-thaw deterioration than wood-fromed decks. Generally, the forms themselves were found to be in good condition when designed for proper deck-surface drainage. /Author/ This article appeared in Transportation Research Record No. 613, Air Sampling, Quality Control, and Concrete.

154124 DA
DEVELOPMENT OF A HYDROPHOBIC SUBSTANCE TO MITIGATE PAVEMENT ICE ADHESION
Ahlborn, GH; Poehlmann, HCJ
Ball Brothers Research Corporation; .P.O. Box 1062; Boulder; Edison; Cincinnati; Colorado; New Jersey; Ohio; 80302; 08817
Dec 1976 218 pp 1976
AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161
REPORT NO.: EPA/600/2-76/242; PB-263653/8ST
CONTRACT NO.: EPA-68-03-0359; Contract
SUBFILE: NTIS; HRIS
The specific problem to which this report is addressed is the development of a hydrophobic substance to mitigate the adhesion of ice to pavement as an alternative to deicing chemicals. The factors involved in evaluating this concept are the following: Economics; safety; environmental impact; coating effectiveness; potential pavement damage. As a result of this program, two coating formulations (exact formulae are given in Chapter 5 of this report), have been identified as showing considerable promise as semi-permanent, hydrophobic, road coatings with reduced ice adhesion. They are: A modified (no pigment) Federal Specification TT-P-115D traffic paint containing a room-temperature-curing silicone rubber (Dow Corning DC732) as a release agent; and a silicone resin waterproofing compound (Dow Corning DRI-SIL-73) combined with the same silicone rubber as above. One major achievement in this program was the discovery of a method for stabilizing the highly reactive silicone rubber in a fluid solution for spraying. Sponsored in part by Municipal Environmental Research Lab., Edison, N.J. Storm and Combined Sewer Section.

153002 DA
NEW CONCEPTS IN BRIDGE DECK WATERPROOFING
Leman, MC
International Bridge, Tunnel & Turnpike Assn, Inc; .Suite 307, 1225 Connecticut Avenue, NW; Washington; D.C.; 20036
Johns-Manville Sales Corporation
pp 5-6 1975
SUBFILE: HRIS
The successful innovation of asbestos asphalt for use as a bridge deck waterproofing agent is reported. Asbestos fibers have been introduced into the basic asphalt mix where they fill the voids of the material. Tests have found this material to be impervious to water and more durable than existing bituminous concrete mixes. This was bourne out by the results of electrical resistance testing which measured the moisture content of water that penetrated the void between the membrane and the bridge deck. This highly efficient water proofing membrane could be placed using standard paving techniques, and causes minimal delays to roadway users as well as reduced job time. This paper was published as part of the Report of the Research Committee Meeting held in Denver, Colorado, August 6-8, 1975.

144218 DA

A FIELD STUDY OF THE PERFORMANCE OF BRIDGE DECK WATERPROOFING SYSTEMS IN ONTARIO

Corkill, JT

Roads and Transportation Association of Canada 1765 St Laurent Boulevard Ottawa Ontario K1G 3V4 Canada

Ontario Ministry of Transportation & Communic, Can

1976 Report pp 76-100 1 Fig. Tabs. 4 Ref.

REPORT NO.: No. 2;

SUBFILE: RTAC; HRIS

In 1971 an investigation of waterproofing systems in use was carried out. The resulting report recommended abandonment of the emulsion membrane waterproofing system and the use of new types of waterproofing methods. The newly developed flexible membrane described as hot rubberized asphalt was adopted for use on the more flexible structures while mastic other waterproofing systems have been used experimentally up to the present. This report discusses the current investigation and field studies of these new waterproofing systems. Annual Conference Proceedings, Calgary, 1975.

144195 DA

THE DESIGN AND CONSTRUCTION OF A MASTIC ASPHALT MEMBRANE FOR WATERPROOFING CONCRETE BRIDGE DECKS IN THE CITY OF CLAGARY

Rodier, CE

Roads and Transportation Association of Canada 1765 St Laurent Boulevard Ottawa Ontario K1G 3V4 Canada

Calgary, City of, Canada

1976 Report pp 43-51 2 Tab.

REPORT NO.: No. 2;

SUBFILE: RTAC; HRIS

Portland Cement concrete spalling and concrete delamination are generally recognized as the most serious and troublesome kinds of bridge deck deterioration. This deterioration results primarily from corrosion forces that radiate from the deck surface, which spall and ultimately expose the reinforcement. Annual Conference Proceedings, Calgary, 1975.

144106 DA

BRIDGE DECK DETERIORATION IN COLORADO

Steere, LB; Tapp, SC; Whalin, WV

Colorado Department of Highways; .4201 East Arkansas Avenue; Denver; Colorado: 80222

Jan 1976 Intrm Rpt. 36 pp 1976

AVAILABLE FROM: National Technical Information Service 5285 Port Royal Road Springfield Virginia 22161

REPORT NO.: FHWA-CO-RD-76-1; PB-259492/AS

CONTRACT NO.: 1475; Contract

SUBFILE: HRIS; NTIS

This is an Interim Report which reviews the capability of the Department of Highways to detect bridge deck deterioration and to take remedial and deterrent action. Department personnel have observed FHWA demonstrations of, and gained experience in the use of, equipment and procedures to evaluate bridge decks. Decks have been repaired using a wide range of materials and processes, from limited patching with both common and unusual or experimental proprietary materials, to extensive removal and replacement of deck concrete. An expanded program of removal of damaged and/or contaminated deck material and replacement with latex modified concrete will be undertaken during the upcoming construction season. Placement of the same material on a new structure also will occur this year. Considerable laboratory and field testing of moisture barrier membranes has been done. Specifications covering fabrication and application of membranes have been published and refined. An extensive retesting program is being carried out to check the continued effectiveness of waterproofing membranes and installation techniques. Research continues in an effort to bring forth economical methods for prevention of bridge deck deterioration and repair of decks already damaged. /FHWA/ Sponsored by Colorado Department of Highways and prepared in cooperation with Department of Transportation, Federal Highway Administration.

138156 DA

WATERPROOF MEMBRANES FOR PROTECTION OF CONCRETE BRIDGE DECKS-LABORATORY PHASE

Van Til, CJ; Carr, BJ; Vallerga, BA

Materials Research and Development, Incorporated

NCHRP Report N165 70 pp Figs. Tabs. Phots. 17 Ref. 3 App. 1976

AVAILABLE FROM: Transportation Research Board Publications Office 2101 Constitution Avenue, NW Washington D.C. 20418

CONTRACT NO.: HR 12-11; Contract

SUBFILE: HRIS; NTIS

This report documents and presents the results of a comprehensive assessment of the protective capabilities of all bridge deck waterproofing membranes known to be available when the project started (1970). An extensive program of laboratory testing supplemented by a limited program of field study led to the selection from an original group of 147 systems, five that appeared most promising for more intensive field evaluation. Materials and construction specifications were prepared for the five systems, in addition to an experimental plan for the field evaluation. The new work is being under taken as a second phase of the study. Research performed by Materials Research and Development, Incorporated. Research sponsored by the American Association of State Highway and Transportation Officials in cooperation with the Federal Highway Administration. See also NTIS/PB-255661/1ST.

128583 DA
POLYPROPYLENE MEMBRANE PROTECTS BRIDGE DECKS AGAINST MOISTURE, SALT DAMAGE
Cooperider, NL
Better Roads VOL. 45 NO. 10 .Oct 1976 pp 32-33 6 Phot. 1976
SUBFILE: HRIS
A non-woven polypropylene black waterproofing fabric applied between asphalt binder and asphalt overlay has been used to prevent deterioration of underlying bridge surfaces. The fabric is available in 75 and 150 inch widths and special order widths, and in rolls of 100 yards. In a Kansas City project, liquid asphalt cement (85 to 100 penetration) using 0.35 gallons per square yard was applied to the bridge deck at a temperature of 375 f. A No. 620 tractor-driven laying machine was used to install the special membrane stips 75 inches wide over the area coated with hot liquid asphaltic cement. A 2 inch asphaltic wearing surface was laid on the mat. The heat from the hot asphaltic wearing surface causes the asphaltic cement binder to saturate the membrane and tack the new overlay. The membrane absorbs approximately two-tenths of the asphaltic cement. This results in a homogeneous mass, binding the wearing surface and the mat to the bridge deck to form a strong barrier to resist moisture. A life of 10 to 20 years is expected for the water barrier. On previous bridge jobs, the membrane was spread and laid manually. A total of 555 resistance readings were taken on the deck: 98.7 percent were over 200,000 ohms; 98.4 percent were over 500,000 ohms; and 95.3 percent were infinite. The barrier system is effective in areas that previously showed reflective cracking and aligator cracking. The system has also helped correct correct deteriorating curb areas.

127960 DA
BRIDGE DECKS
Godfrey, KA, Jr
ASCE Civil Engineering VOL. 45 NO. 8 .Aug 1975 pp 60-65 2 Fig. 1 Tab. 10 Phot. 1975
SUBFILE: HRIS
This is the second of two articles on short span bridges. The first, in the July issue, described cost-cutting ideas in design. This article focuses on bridge decks: expansion joints, open steel grid decking, timber decks, the Iowa Method of deck construction (a very low permeability concrete overlay), waterproof membranes, plastic and wax sealing of deck concrete, and various approaches to protecting the rebars from corrosion-galvanizing, epoxy coating, stainless cladding, etc. /ASCE/

127783 DA
FACELIFT UPDATES VETERAN RC BRIDGE
IPC Building and Contract Journals, Limited
Contract Journal VOL. 265 NO. 4995 May 1975 p 21 2 Phot.
SUBFILE: TRRL; IRRD; HRIS
One of the earliest reinforced concrete bridges built in Great Britain, the seventy year old Tuckton Bridge over the River Stour in Christchurch is being reinstated to its original design. The face of the existing beams, columns and ribs is being reduced by 6mm and where necessary down to sound concrete. Much of the concrete was in bad shape, largely the result of old age, and in some areas had spalled away, leaving the reinforcing bars exposed. Quickmast 481 dispersible epoxy resin with a waterproof sand and cement render in multiple coats 12.5mm thick is being used. Where the reinforcement is exposed, it is being wire brushed and treated with epoxy resin. The new finish will be of fine wood float. During the operation the bridge is not being closed but the lane width is restricted. The deck has been re-surfaced. Costs are quoted.

126102 DA
THE MARTIGUES VIADUCT (FRANCE)
Poirson, C; Foucriat, JC
Centre Belgo-Luxem d'Information de l'Acier
Acier/Stahl/Steel N6 Jun 1972 pp 257-261 3 Fig. 5 Phot. 1 Ref. French
SUBFILE: TRRL; IRRD; HRIS
The Marseille-Fos Motorway crosses the Caronte Canal by means of a structure linked to two interchanges. It comprises a central metal structure between two access concrete viaducts. The length of the whole structure is 874m, the dual carriageway is 14m wide. Details are given of the special installation method used. The access viaducts are in prestressed concrete; they measure 314.50m and 252.50m, the spans being 65m. The piers are made of reinforced concrete and were built with the help of sliding forms. The central part is a metal 300-m long bridge with inclined portal frames. The deck consists of a rectangular box girder, on which rests the slab, which is 30m long. The slab is of the orthotropic plate type. The prefabrication and the transport of the metal units are described (some were transported by rail, others by water), together with the assembling, the lifting of the metal deck, the positioning of the portal frames, safety devices, carriageway joints, painting, surfacing, asphaltic varnish, and polymer-bitumen waterproofing course. /TRRL/

102968 PR
BRIDGE DECK DETERIORATION AND RESTORATION
INVESTIGATORS: Korfhage, GR; Halverson, AD
PERFORMING ORG: Minnesota Department of Transportation 408 Transportation Building St Paul Minnesota 55155
SPONSORING ORG: Minnesota Department of Transportation; Minnesota Local Road Research Board; Federal Highway Administration Department of Transportation
CONTRACT NO.: 319; HP&R
SUBFILE: HRIS
PROJECT START DATE: 7212
PROJECT TERMINATION DATE: 8112
A NUMBER OF MATERIALS WERE PLACED ON THE DECKS OF HIGHWAY BRIDGES TO SERVE AS WATER PROOFING SYSTEMS TO PREVENT WATER AND CHLORIDES FROM REACHING THE REINFORCING STEEL. INCLUDED ARE A LATEX-MODIFIED CONCRETE WEARING SURFACE, LOW SLUMP CONCRETE SURFACE, PREFORMED RUBBER-LIKE SHEET MEMBRANES WITH ASPHALTIC CONCRETE SURFACE COURSE, AND MEMBRANE-FORMING LIQUID MATERIALS ALSO SURFACED WITH ASPHALT CONCRETE. EPOXY COATED REBARS AND CATHODIC PROTECTION HAVE ALSO BEEN USED. EVALUATION IS BEING MADE BY LONGTERM OBSERVATION OF SERVICE PERFORMANCE AND BY ELECTRICAL RESISTIVITY MEASUREMENTS.

102944 PR
EVALUATION OF WATERPROOFING MEMBRANES ON CONCRETE DECKS
INVESTIGATORS: Azevedo, WV
PERFORMING ORG: Kentucky Transportation Research Program Kentucky University, College of Engineering 533 South Limestone Lexington Kentucky 40508
SPONSORING ORG: Kentucky Department of Transportation Bureau of Highways
SUBFILE: HRIS
PROJECT START DATE: 7308
PROJECT TERMINATION DATE: 7906
THE OBJECTIVES ARE TO MONITOR CONSTRUCTION OF REINFORCED CONCRETE BRIDGE DECKS CONTAINING WATERPROOFING MEMBRANES AND TO MONITOR, EVALUATE, AND DOCUMENT THEIR PERFORMANCE. ELECTRICAL RESISTANCE MEASUREMENTS WILL BE MADE UPON COMPLETION OF CONSTRUCTION AND PERIODICALLY THEREAFTER IN ORDER TO DETERMINE THE EFFECTIVENESS OF EACH MEMBRANE.

099303 DA
CONTINUING FIELD EVALUATION OF MEMBRANE WATERPROOFING FOR CONCRETE BRIDGE DECKS
Carroll, RJ
Ohio Department of Transportation; .25 South Front Street; Columbus; Ohio; 43216
Jan 1975 Intrm Rpt. 49 pp 2 Fig. 3 Tab. 1975
CONTRACT NO.: Job No. 14270(0)
SUBFILE: HRIS
This report presents the results of electrical resistivity testing, made in the field and on core samples, of membrane installations. It summarizes the results of such testing on the different products used in Ohio with lengths of service up to seven years, discusses the condition and apparent effectiveness of existing installations; includes recommendations as to the use of specific membranes and comments on installation. Prepared in cooperation with FHWA.

097751 DA

CRACKING OF VOIDED POSTTENSIONED CONCRETE BRIDGE DECKS; THE MACOWAN BRIDGE TEST; PROTOTYPE FOR VOIDED POSTTENSIONED CONCRETE BRIDGE DECKS; MODEL STUDIES FOR VOIDED POSTTENSIONED CONCRETE SLAB BRIDGES; FINITE ELEMENT COMPUTER PROGRAM FOR ANALYSIS OF PLATES WITH BENDING AND IN-PLANE STRESSES

Csagoly, P; Holowka, M; Campbell, TI; Meades, P; Green, R; McNeice, GM

Ministry of Transportation and Communications, Can; .Research and Development Division; Downsview; Ontario M3M 1J8 : Canada

Jan 1975 49 pp 50 Fig. 21 Ref. 1975

REPORT NO.: RR193, 194, 195, 196;

SUBFILE: HRIS

This study of concrete cracking, included monitoring of deformations and temperature variations of prototype bridges, acrylic modelling of two typical prototype bridges, building of an automated finite element computer program, surveying and coring a large number of bridge decks, chemical analysis for chloride content, long-term testing of voided, micro-concrete models, constructing a representative cross-sectional slice of a prototype bridge, and using rigid foams as void-formers. The temporary measure of applying a waterproofing membrane covered with an asphaltic concrete wearing surface can promote deterioration in newly-constructed decks. With the use of appropriate 2- and 3-dimensional analysis, it is possible to achieve crack control by applying adequate transverse, skew and/or fanned prestressing. The second volume of this report describes field tests carried out on an underpass structure which has a 3-span continous, posttensioned concrete deck which was cast in-situ and contains longitudinal circular voids. Measurements are presented for various parameters from the time of placing the concrete, through prestressing, and also for the deck subjected to static and dynamic loading. Local strains, internal cracking and the propagation of cracks are discussed. The third section of this report discusses design, construction, instrumentation, and loading of scale models of a 3-span, straight slab bridge and a two span, skew slab bridge. Both the models and instrumentation responded elastically to loads. Conclusions (based on this study) are presented which relate to transverse tensile stresses, transverse prestress and model structures which behaved as orthotropic slabs. The forth volume of this report covers the computer stress analysis of continuous voided, posttensioned concrete bridge decks performed to explain the appearance of cracks around the area of the voids after construction. The limitations of the 2-dimensional analysis including bending and in-plane stresses, were evident in end anchorage zones, at support points and in the wing area of the decks. The system of finite element programs are outlined here which require approximately 220k bytes of core stone and which was developed for the IBM 360/75 computer, compiled under Fortran H Level 2. This document contains a series of four reports.

081122 DA

AN EVALUATION OF THE EFFECTIVENESS OF MEMBRANE WATERPROOFING FOR CONCRETE BRIDGE DECKS

Henry, JK

Ohio Department of Transportation

Ohio Transportation Engineering Conf. Proceedings VOL. 28 .Apr 1974 pp 91-101 Figs. Tabs. 1974

SUBFILE: HRIS

The assessment is reported of the effectiveness of waterproofing membranes in Ohio (almost exclusively of the liquid-applied type) which involved the laboratory testing of most all membranes and the collection of data on all new field installations of both liquid and sheet membranes. The effect of construction practices was noted, and the use of a sand asphalt mix between the membrane and the surface course of asphaltic concrete was studied. Samples of membranes were made and tested for loss of impermeability by a modified version of the resistivity testing apparatus. The impermeability of membranes in the field was measured by a recently developed electrical resistivity method. Evaluatory comments are made on liquid-applied membrane systems such as Firestone and Goodyear, Marbilistic liquid neoprene, etc., and sheet membranes. Two built-up systems (a coal-tar emulsion with fiberglass reinforcing mesh and a Type B membrane) and a rubberized sand-asphalt system are also discussed. Presented at the Twenty-Eighth Annual Ohio Transportation Engineering Conference. Formerly: Ohio Highway Engineering Conference

APPENDIX B

WSDOT SPECIFICATION FOR MEMBRANE WATERPROOFING CONCRETE BRIDGE DECKS

MEMBRANE WATERPROOFING (DECK SEAL)
(December 13, 1982)

Description

This work shall consist of furnishing and placing an approved waterproofing membrane over a properly prepared concrete bridge deck prior to placing the asphalt concrete overlay, in accordance with these specifications, and in reasonably close conformity with the plans or as directed by the Engineer.

The waterproofing membrane for this project shall be selected by the Contractor from one of the following systems:

System A

A factory laminated sheet composed of either suitably plasticized coal tar or rubberized asphalt reinforced with a polypropylene fabric and primed in accordance with the manufacturer's recommendations.

System B

A hot-applied, rubberized elastomeric membrane with primer if required by the manufacturer.

System C

A hot-applied reclaimed rubber-asphalt membrane.

Preparation of Concrete Deck

The entire deck and the sides of the curb to the height of the asphalt overlay shall be essentially free of all foreign material such as dirt, grease, etc. Prior to applying the primer or liquid membrane, all dust and loose material shall be removed from the deck with compressed air. Any surface defects such as spalled areas, cracks, protrusions, etc., that will decrease the effectiveness of the membrane by puncturing, stretching, etc., shall be corrected prior to application of the membrane.

Weather and Moisture Limitations

Work shall not be done during wet weather conditions, nor when the deck and ambient air temperatures are below 50 degrees F. The deck shall be surface-dry at the time of the application of the primer or liquid membrane.

The Engineer may order work to be suspended in accordance with section 1-08.6 of the standard specifications because of the above weather and moisture limitations.

New Concrete Areas

Any area of the deck that has less than 28 day old concrete shall be allowed to cure for a period of time recommended by the membrane manufacturer or as ordered by the Engineer before application of the membrane.

Concrete Protection
The Contractor shall use care to protect all concrete surfaces from damage. Any damage to exposed surfaces shall be repaired at the Contractor's expense.

Membrane Application
The primer and membrane waterproofing shall extend from the roadway deck up onto the curb face the thickness of the asphalt overlay. Special care shall be used at the curb face to see that the membrane adheres to the concrete.

The Contractor shall not begin application of membrane waterproofing deck seal to the bridge until he has demonstrated, to the satisfaction of the Engineer, that all labor, equipment, and materials necessary to apply the membrane and asphalt concrete overlay are either on hand or readily available to complete the work in a timely manner.

Membrane Protection
The membrane material shall be protected from damage due to the paving operations. The method of membrane protection for Membrane Systems A and B shall be as recommended by the manufacturer of the membrane system and approved by the Engineer. The method of membrane protection for Membrane System C shall be as specified under Membrane System C.

No traffic or equipment except that required for the actual waterproofing and paving operations will be permitted to travel or rest on the membrane waterproofing until it is covered by the asphalt overlay.

Asphalt Concrete Overlay
The membrane manufacturer's recommendations shall be thoroughly considered in the application of the asphalt concrete overlay particularly as to the type of paving machine, laydown temperature of the asphalt concrete, protection of membrane while paving, rolling temperature and technique and other items unique to each membrane. Differences in application procedure shall be resolved by the Engineer and his decision shall be final. Vibratory rollers shall not be used on bridge decks.

Evaluating Waterproofing Effectiveness
When, in the Engineer's judgment, it is indicated that the completed sections of the waterproofing membrane should be evaluated for waterproofing effectiveness prior to application of membrane protection, testing by WSDOT Test Method No. 413A shall be performed. Any portion of the membrane found to have a resistance reading below 100,000 ohms shall be repaired. Those membranes which provide less than 70 percent readings above 250,000 ohms shall be replaced or, at the option of the Contractor, repairs may be made to bring the membrane to the acceptance level.

After completion of the asphalt overlay, a final evaluation of the waterproofing effectiveness of the membrane/pavement system shall be made in accordance with WSDOT Test Method No. 413A. The acceptance standards for the pavement/membrane system shall be 70 percent readings above 250,000 ohms and no single reading below 100,000 ohms. Those areas requiring repair or replacement to meet acceptance standards shall be corrected as directed by the Engineer.

The testing will be conducted by state forces.

Membrane System A

Materials

Primer
The primer used to bond membrane to deck and to seal seams and patches shall be a water resistant adhesive compatible with the membrane. The primer shall be of suitable consistency for application by brush, roller or spray without further dilution.

Membrane
The membrane shall be factory-laminated sheet composed of either suitably plasticized coal tar or rubberized asphalt reinforced with polypropylene fabric. It shall be manufactured free from blemishes, discontinuities and other defects. The membrane shall be supplied in rolls, having a minimum width of 36 inches, and shall conform to the following requirements:

Thickness:	65 mils minimum
Pliability (1):	No cracks

(1) Place a 4 inch by 1 inch membrane specimen in a -10 degree F cold chamber for 2 hours. While still in the cold chamber, bend the specimen 180 degrees over a 1 inch radius mandrel. Remove specimen from cold chamber and inspect for cracks.

Application
The primer shall be applied to the cleaned concrete surfaces at the rate and according to the procedure recommended by the membrane manufacturer. All surface to be covered by the membrane shall be thoroughly and uniformly coated with primer. Precautionary measures shall be taken to assure that pools and thick layers of primer are not left on the deck surface to scum over. Drying time prior to applying the membrane shall normally be as recommended by the manufacturer, however, the membrane shall not be applied until substantially all volatile material has dissipated from the primer.

The prefabricated membrane shall be applied to the primed curb and bridge deck surfaces by either hand methods or mechanical applicators. The membrane shall be placed in such manner that a shingling effect will be achieved and that any water which accumulates will drain toward the curb and the drain pipes. Each strip will be overlapped a minimum of 4 inches. An adhesive or a wide tipped torch to cause tackiness shall be used, if necessary, to assure a good seal of the joints. Hand rollers or other satisfactory pressure apparatus shall be used on the applied membrane to assure firm and uniform contact with the primed concrete surfaces.

Any torn or cut areas, or narrow overlaps, shall be patched using a satisfactory adhesive and by placing sections of the membrane over the defective area in such a manner that the patch extends at least 6 inches beyond the defect. The patch shall be rolled or firmly pressed onto the surface.

The fabric shall be neatly cut and contoured at all joints as directed by the Engineer.

After the membrane waterproofing has been completed, the membrane shall be cut with 2 right angle cuts at all deck drain pipes. The cuts shall be made to the inside diameter of the drain pipes, after which the corners of the membrane waterproofing shall be turned down into the drains and laid in a coating of asphalt binder.

Membrane System B

Materials

Primer
The primer, if required, shall meet the manufacturer's recommendations.

Membrane
The membrane shall meet the following requirements:

Viscosity, SSF at 350 degrees F	ASTM E 102	950-1350
Softening Point, degrees F	ASTM D 2398	165 Min.
Adhesion, PSI	ASTM D 429(1)	15 Min.
Cold Bend Test, Minus 10 degrees F	(2)	No cracks
Compatibility with Asphalt		Complete

(1) Adhesion ASTM D 429, Method A(Modified). This is a tension test of vulcanized rubber to steel. Coat the surfaces of the 2 metal plates described in the procedure with an epoxy resin of at least 2000 PSI tensile

strength. Stand the coated ends on Ottawa sand (ASTM C 109). Apply a pressure of 10 lbs. for a minimum of 8 hours to insure adequate bedding of the sand in the resin. Brush all loose particles from the treated metal surface and coat each with 0.3 gram of primer.

Cover the bottom of a cylindrical thin film oven test pan (ASTM D 1754) with a release paper such as Technipeel No. 985 made by the Brown Paper Company of Kalamazoo, Michigan. Any release paper that retains its release properties after use is satisfactory.

Pour 50 grams of membrane (350 - 375 degrees F) into the release-treated pan. Allow the membrane to cool to ambient temperature. Remove it from the pan and cut circular sections to fit the metal plates of discs coated as above.

Fit the circular section of membrane on one metal disc and place the other metal disc over it. Put this sandwich in a 140 degree F oven. Place a 1 kg weight on it and leave in the oven for 2 hours.

Test in accordance with ASTM D 429, Method A, and calculate adhesion as total load at failure divided by area of adhered surface, whether failure occurred at bonded surface or within the membrane material.

(2) Cold Bend Test. Pour 50 grams of membrane into a container, as described for the adhesion test. Allow it to cool to ambient temperature and remove from the pan.

Dust both sides lightly with talc to prevent stickiness.

Place the specimen in a -10 degree F cold chamber for 2 hours. While still in the cold chamber, bend the specimen 180 degrees over a radius not to exceed one inch.

Remove the membrane from the cold chamber and check for cracks. Only material which shows no cracks will be considered satisfactory.

Application

Primer
The primer, if required, shall be applied to the pavement as specified by membrane manufacturer.

Membrane Application
If the primer has become contaminated, the pavement shall be cleaned and a new primer applied and allowed to cure before the membrane is applied.

The membrane material shall be heated in accordance with the manufacturer's recommendations. In order to insure against overheating, a double-boiler type heater shall be used and the membrane material shall be circulated or agitated during the heating process.

The membrane shall be applied to the clean, dry (primed) surface at a nominal rate of 0.5 gallon per square yard and in accordance with the manufacturer's recommendations as to application temperatures.

Placement of the asphalt concrete wearing surface shall be done in accordance with the recommendations of the coating manufacturer.

Membrane System C

Materials
The asphalt shall be paving grade asphalt meeting the requirements for AR-1000 as specified in the Uniform Pacific Coast Asphalt Specifications or it shall be AR-2000W or AR-4000W conforming to section 9-02.1(4) of the standard specifications.

The granulated rubber shall meet the following requirements:

It shall be free from fabric, wire, or other contaminating materials except that up to 4 percent of calcium carbonate may be included to prevent particles from sticking together. The gradation of the rubber shall meet one of the following and shall be at the option of the Contractor:

1. When only ground vulcanized rubber is used, it shall be at least 95 percent passing the No. 16 sieve and not more than 10 percent shall pass the No. 25 sieve.

2. If powdered, reclaimed devulcanized rubber is used, approximately 40 percent shall be added to 60 percent ground vulcanized rubber scrap

and the total blend shall meet the following grading:

Sieve	% Passing
No. 8	100
No. 30	60 - 80
No. 50	15 - 40
No. 100	0 - 15

Mixing Asphalt and Rubber

The asphalt and rubber shall be combined as rapidly as possible in the proportions of a minimum of 2 pounds of rubber to 1 gallon of asphalt (standard @ 400 degrees F) then held for such a time and temperature that the consistency of the mix approaches that of a semi-fluid material. The temperature of the asphalt shall be between 400 degrees F and 450 degrees F. The use of up to 9 percent diluent to assist in the mixing of the rubber and the spray application of the mixture will be permitted. If a diluent is used, it shall have a boiling point of at least 350 degrees F. After reaching the proper consistency, application shall proceed immediately, and in no case shall the material be held at temperatures in excess of 350 degrees F for more than 1 hour after reaching that point.

The method and equipment for combining the rubber and asphalt shall be so designed and accessible that the Engineer can readily determine the percentages, by weight, of each of the 2 materials being incorporated into the mixture.

Equipment

The equipment used for mixing and spreading the asphalt and rubber shall be a self-powered pressure distributor equipped with a separate power unit, distributing pump capable of pumping the specified material at the specified rate through the distributor tips, and equipment for heating the bituminous material. The distribution bar on the distributor shall be fully circulating with nipples and valves so constructed that they are bathed in the circulating asphalt to the extent that the nipples will not become partially plugged with congealing asphalt upon standing. Distributor equipment shall include a tachometer, pressure gauges, volume measuring devices, and a thermometer for reading temperatures of tank contents. The spray bars on the distributor shall be controlled by a bootman riding at the rear of the distributor in such a position that operation of all sprays is in full view and accessible to him for controlling spread widths.

Application

The application rate of the hot asphalt-rubber mixture shall be 0.55 ± 0.10 gallon per square yard uniformly applied.

All transverse joints shall be made by placing building paper over the ends of the previous applications, and the joining application shall start on the building paper used. Once the application process has progressed beyond the paper used, the paper shall be removed and disposed of to the satisfaction of the Engineer. If the Contractor can demonstrate the ability to produce satisfactory transverse joints without paper, no paper will be required as long as the joints remain satisfactory. Any unsatisfactory joint shall be repaired at the Contractor's expense.

Membrane_Protection
Prior to overlaying with asphalt concrete, the asphalt-rubber mixture shall be covered with fabric as specified below:

The fabric shall be a polypropylene material having the following properties:

Tensile strength, either direction, min. (ASTM Method D 1682, Grab Method)	47 lbs.
Weight, oz./sq. yd.	3 - 6
Width, inches	36 min.

The fabric shall be aligned and carefully rolled and/or broomed into the asphalt-rubber mixture. Rolling and/or brooming the fabric into the asphalt should be accomplished in such a way that any air bubbles which form under the fabric will be removed. This can best be accomplished by brooming from the center of the fabric toward the outer edges. Initial alignment is very important since the fabric direction cannot be changed appreciably without causing wrinkles. If the alignment of the fabric must be changed, the fabric shall be cut and realigned overlapping the previous material and proceeding as before. All joints shall be overlapped a minimum of one inch.

After the membrane waterproofing has been completed, the fabric shall be cut around the top inside of the frame of inlets and laid in a coating of asphalt binder.

Measurement and Payment

Membrane waterproofing will be measured by the square yard. The area to be measured will be the area of the bridge deck and curb which is satisfactorily sealed and accepted.

The unit contract price per square yard for "Membrane Waterproofing (Deck Seal)" shall be full compensation for furnishing all labor, materials, tools, equipment and incidentals, for doing the work involved in cleaning the surfaces to be sealed and in applying the membrane waterproofing complete in place in accordance with the requirements of these specifications, the standard specifications and as directed by the Engineer. The price paid shall include repairing any damaged or defective waterproofing membrane and damaged asphalt overlay.

Payment

The unit contract price per square yard for "Membrane Waterproofing (Deck Seal)" shall be full compensation for furnishing all labor, materials, tools, equipment and incidentals, for doing the work involved in cleaning the surfaces to be sealed and in applying the membrane waterproofing complete in place in accordance with the requirements of these specifications, the standard specifications and as directed by the Engineer. The price paid shall include repairing any damaged or defective waterproofing membrane and damaged asphalt overlay.

APPENDIX C

WSDOT BRIDGE INFORMATION CARDS
REGARDING BRIDGES 285/10, 82/106 AND 5/650 W

Including:

general card, railroad grade separation,
steel spans, concrete spans

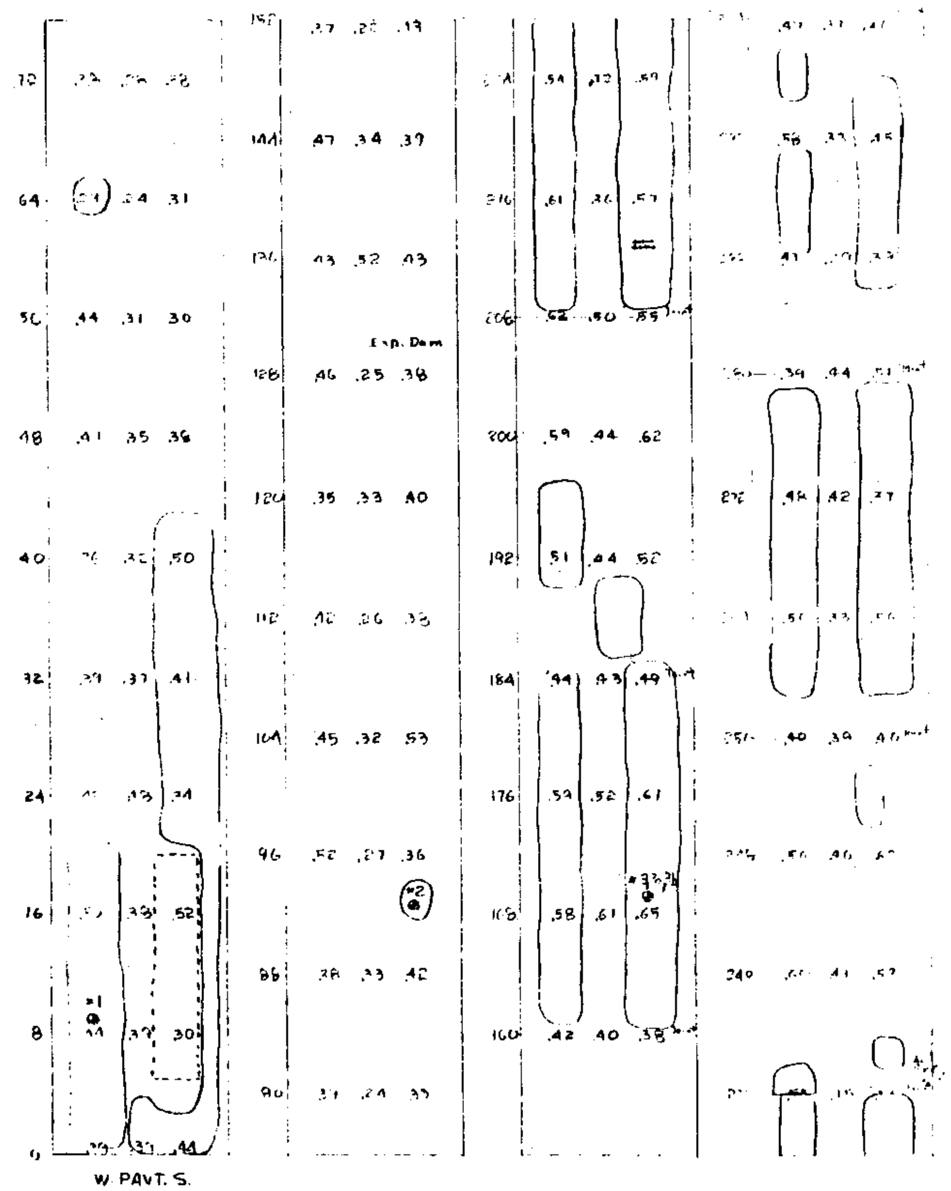
Available from:

Washington State Department of Transportation
Bridge and Structures Branch
Olympia, WA 98504

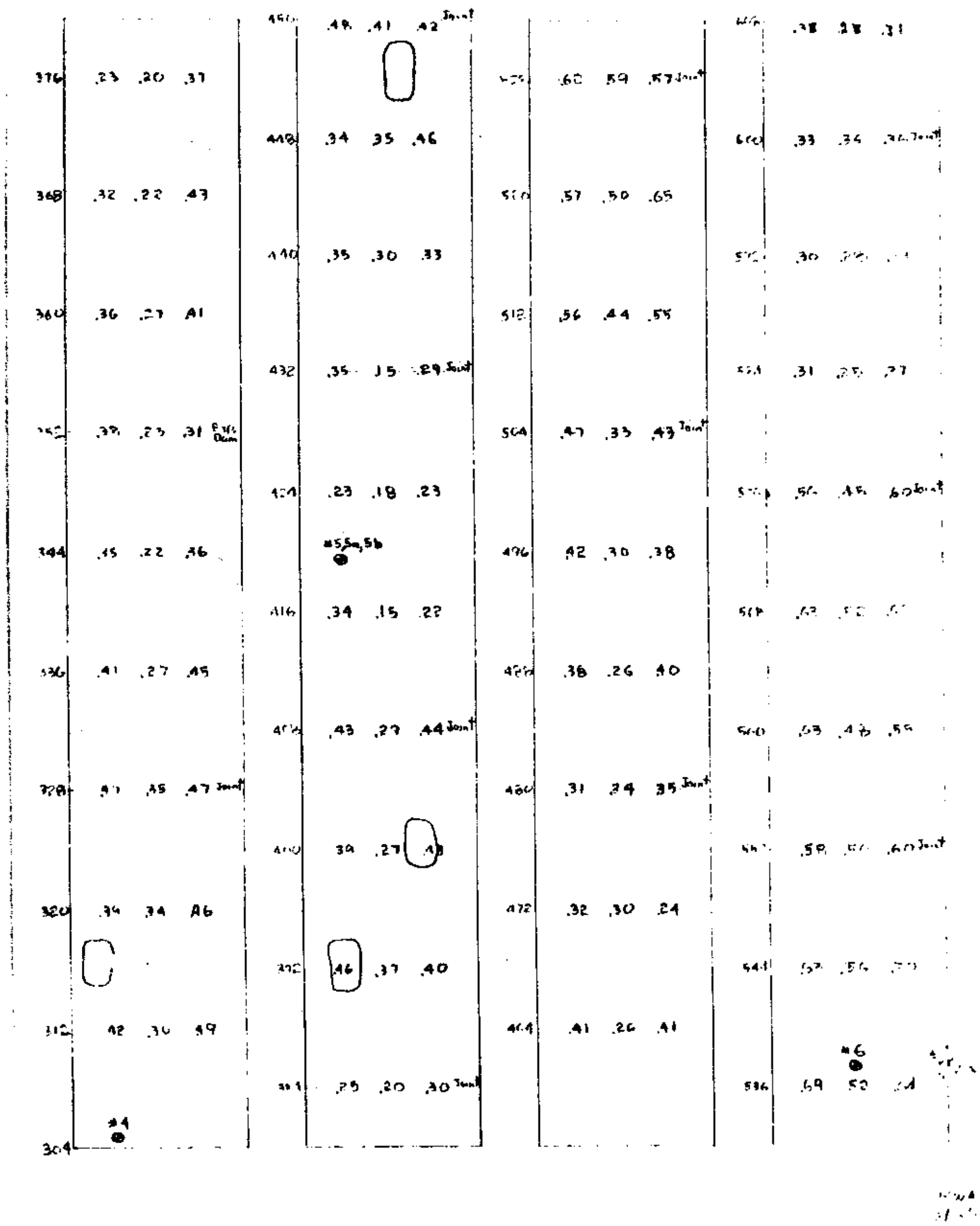
APPENDIX D

RESULTS OF A WSDOT DETAILED DECK CONDITION SURVEY CONDUCTED ON BRIDGES 285/10 AND 5/650 W IN 1975 AND 1974, RESPECTIVELY

Bridge No. 2/250 Columbia River EB Outside Lane

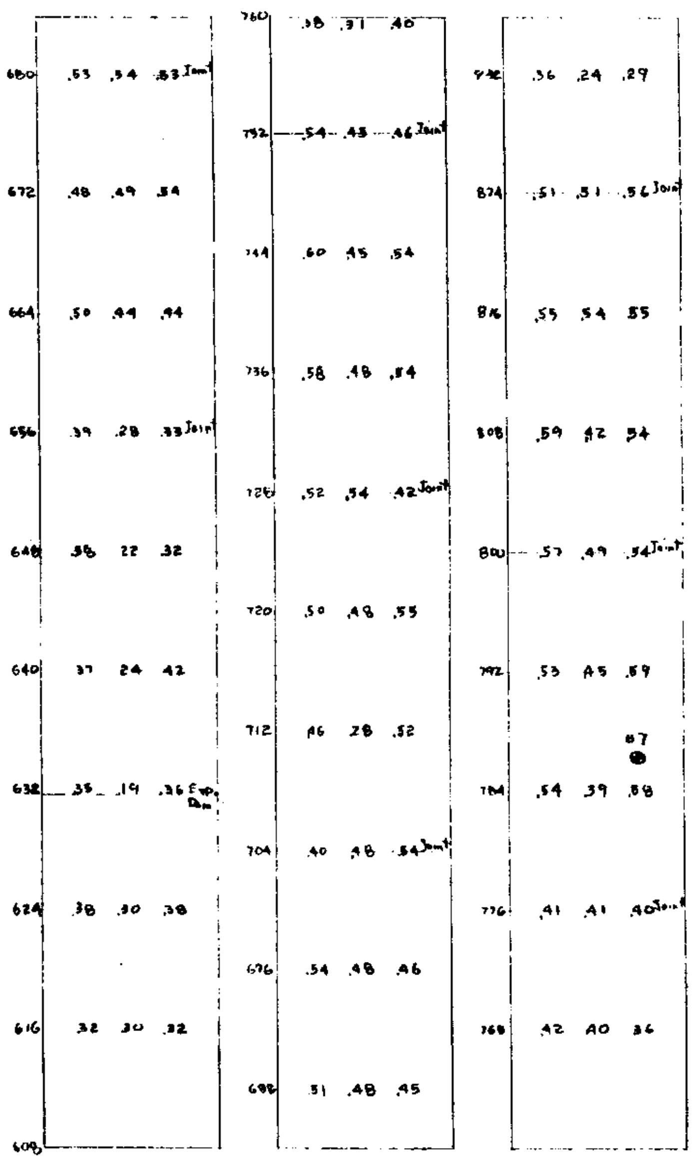


2/250 Columbia River



2/250 Columbia River

Sheet 3 of 3



Delamination =
Exposed Rebar =
Patch =

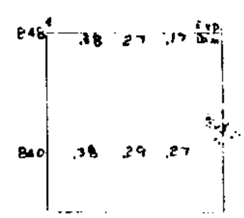
Core location = #X

Scale = 2' 2'

Chloride Content

# 1	1.16 #/yd³
# 2	
# 3	1.50 #/yd³
# 3a	0.80 #/yd³
# 3b	.49 #/yd³
# 4	
# 5	
# 5a	
# 5b	
# 6	3.45 #/yd³
# 7	1.53 #/yd³

* Not possible to do entire bridge due to traffic control limitations.



Bridge No. 5/050 W Ebey Slough Inside Lane B

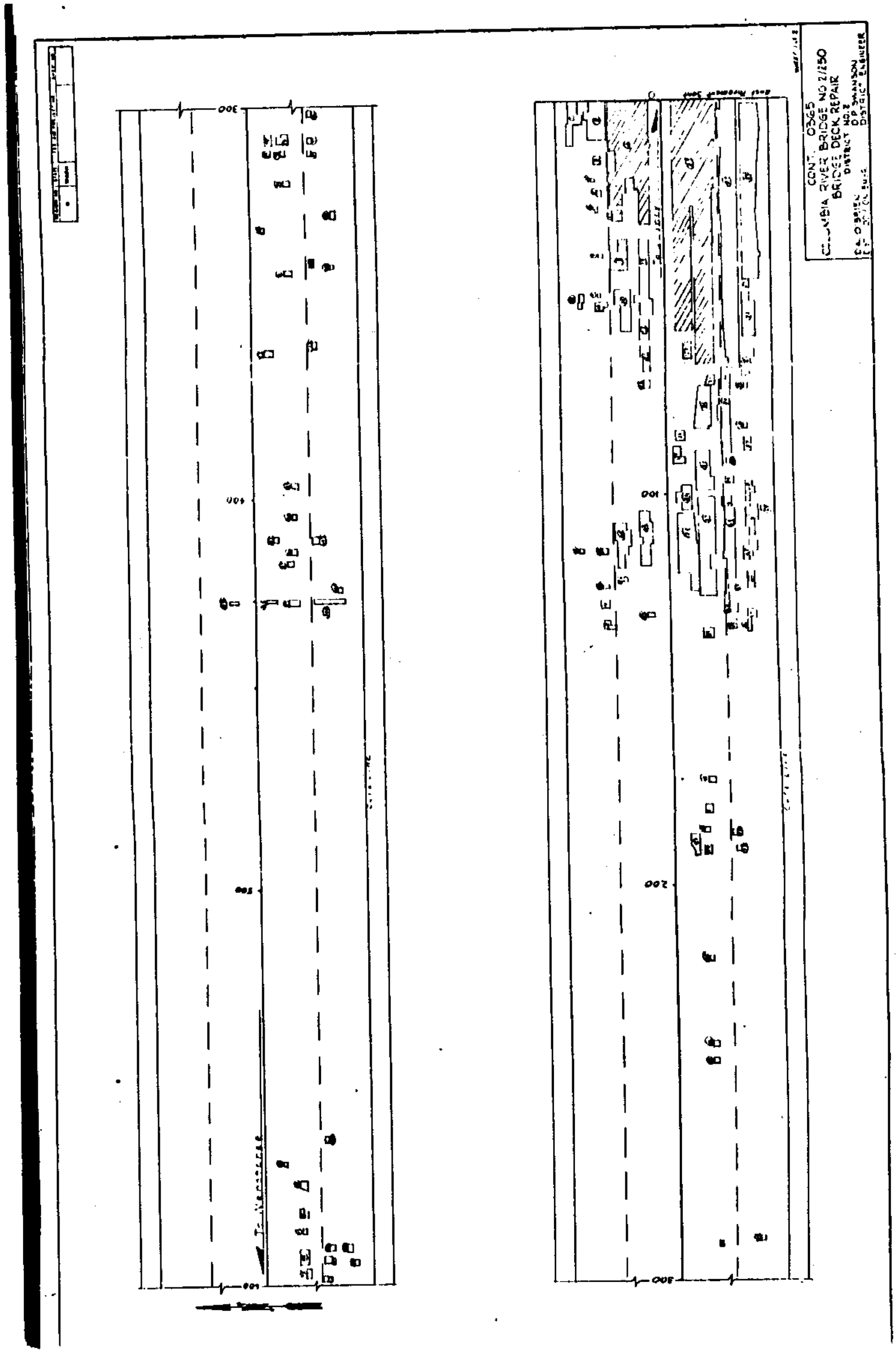
Core Locations				Chloride Content
# 1	100',	10' Rt	RWP	0.66 #/yd³
# 2	250',	5.5' Rt	LWP	
# 3	400',	7.5' Rt	LWP	1.08 #/yd³
# 4	510',	9' Rt	RWP	
# 5	665',	8.5' Rt	LWP	1.28 #/yd³
# 6	820',	5' Rt	-	
# 7	990',	16' Rt	RWP	2.31 #/yd³
# 8	1190',	6' Rt	LWP	
# 9	1400',	12.5' Rt	RWP	2.24 #/yd³
# 10	1620',	6.5' Rt	LWP	

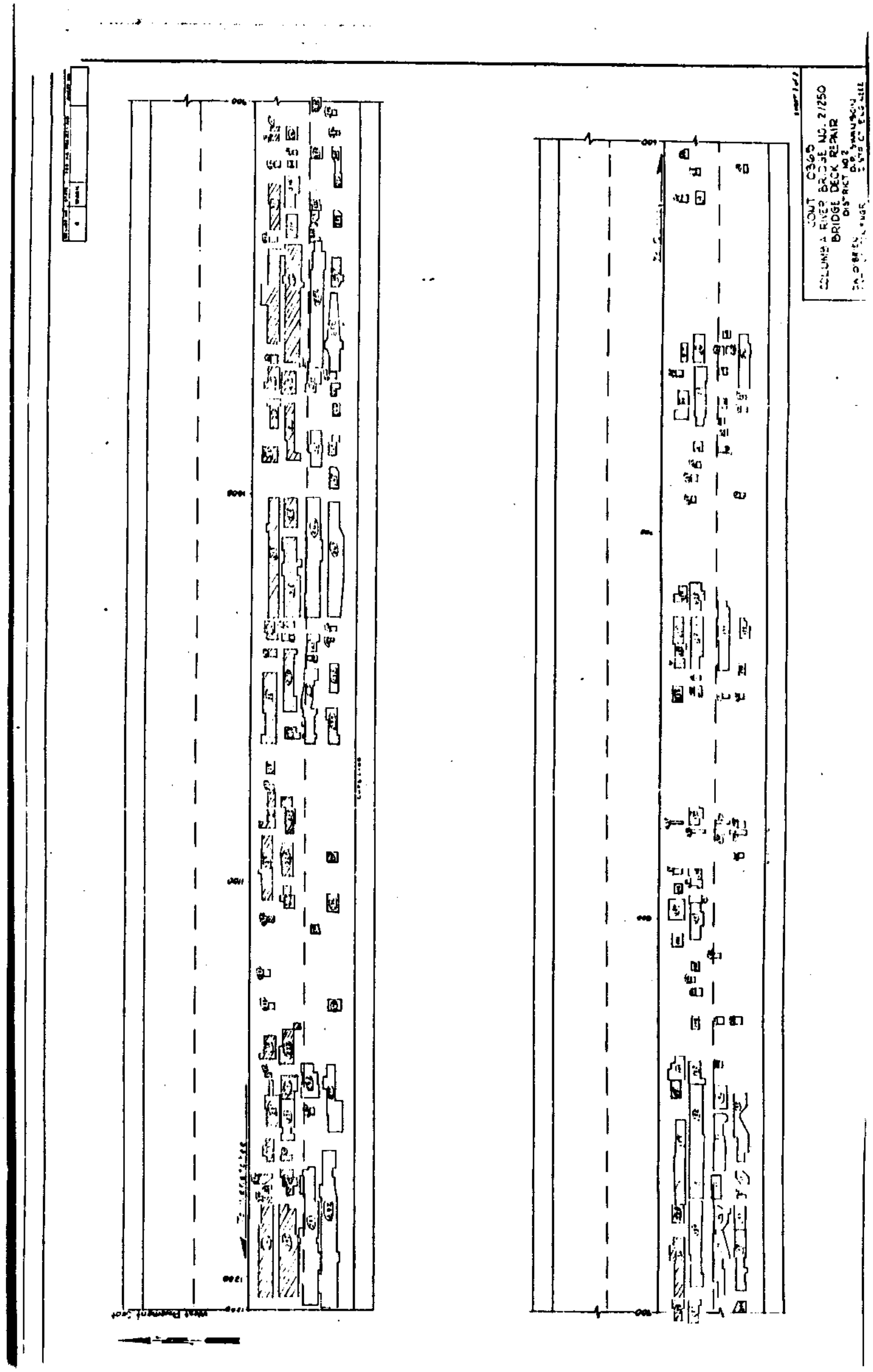
Distances are measured from the north pavement seat

Offsets are measured from the curb — there is a 5' shoulder between the curb and the inside lane

APPENDIX E

LOCATION OF REPAIRED AREAS ON THE DECK OF BRIDGE 285/10 PRIOR TO WATERPROOFING IN 1976


CONT. 0365
RIVER BRIDGE NO 2/250
BRIDGE DECK REPAIR
DISTRICT NO.2
D.P. SWANSON
DISTRICT ENGINEER


BRIDGE DECK REPAIR

APPENDIX F

ASTM TESTING PROCEDURES

Standard Test Method for HALF CELL POTENTIALS OF REINFORCING STEEL IN CONCRETE[1]

This Standard is issued under the fixed designation C 876; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

1. Scope

1.1 This method covers the estimation of the electrical half cell potential of reinforcing steel in concrete, for the purpose of determining the corrosion activity of the reinforcing steel. This method may be used advantageously to assess the relative extent and progress of corrosion of steel embedded in field and laboratory concrete members. This method is applicable to members regardless of their size or the depth of concrete cover over the reinforcing steel.

2. Significance and Use

2.1 The results obtained by the use of this method should not be considered as a means for estimating the structural properties of the steel or of the reinforced concrete member.

2.2 This method is limited by electrical circuitry. A concrete surface that has dried to the extent that it is a dielectric and surfaces that are coated with a dielectric material will not provide an acceptable electrical circuit. The basic configuration of the electrical circuit is shown in Fig. 1.

3. Apparatus

3.1 The testing apparatus consists of the following:

3.1.1 *Half Cell:*

3.1.1.1 A copper-copper sulfate half cell (Note 1) is shown in Fig. 2. It consists of a rigid tube or container composed of a dielectric material that is nonreactive with copper or copper sulfate, a porous wooden or plastic plug that remains wet by capillary action, and a copper rod that is immersed within the tube in a saturated solution of copper sulfate. The solution shall be prepared with reagent grade copper sulfate crystals dissolved in distilled or deionized water. The solution may be considered saturated when an excess of crystals (undissolved) lies at the bottom of the solution.

3.1.1.2 The rigid tube or container shall have an inside diameter of not less than 1 in. (25 mm); the diameter of the porous plug shall not be less than 1/2 in. (13 mm); the diameter of the immersed copper rod shall not be less than 1/4 in. (6 mm), and the length shall not be less than 2 in. (50 mm).

3.1.1.3 Present criteria based upon the half cell reaction of $Cu \rightarrow Cu^{++} + 2e$ indicate that the potential of the saturated copper-copper sulfate half cell as referenced to the hydrogen electrode is −0.316 V at 72°F (22.2°C). The cell has a temperature coefficient of about 0.0005 V more negative per °F for the temperature range from 32 to 120°F (0 to 49°C).

NOTE 1—While this method specifies only one type of half cell, that is, the copper-copper sulfate half cell, others having similar measurement range, accuracy, and precision characteristics may also be used. In addition to copper-copper sulfate cells, calomel cells have been used in laboratory studies. Potentials measured by other than copper-copper sulfate half cells should be converted to the copper-copper sulfate equivalent potential. The conversion technique is described in most physical chemistry or half cell technology text books.

[1] This method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.03 15 on Methods of Testing Concrete for Resistance to Weathering.

Current edition approved Dec. 30, 1977. Published February 1978.

3.1.2 *Electrical Junction Device*—An electrical junction device shall be used to provide a low electrical resistance liquid bridge between the surface of the concrete and the half cell. It shall consist of a sponge or several sponges pre-wetted with a low electrical resistance contact solution. The sponge may be folded around and attached to the tip of the half cell so that it provides electrical continuity between the porous plug and the concrete member.

3.1.3 *Electrical Contact Solution*—In order to standardize the potential drop through the concrete portion of the circuit, an electrical contact solution shall be used to wet the electrical junction device. One such solution is composed of a mixture of 95 ml of wetting agent (commercially available wetting agent) or a liquid household detergent thoroughly mixed with 5 gal (19 litre) of potable water. Under working temperatures of less than about 50°F (10°C), approximately 15 % by volume of either isopropyl or denatured alcohol must be added to prevent clouding of the electrical contact solution, since clouding may inhibit penetration of water into the concrete to be tested.

3.1.4 *Voltmeter*—The voltmeter shall have the capacity of being battery operated and have ±3 % end-of-scale accuracy at the voltage ranges in use. The input impedance shall be no less than 10 million Ω when operated at a full scale of 100 mV. The divisions on the scale used shall be such that a potential difference of 0.02 V or less can be read without interpolation.

3.1.5 *Electrical Lead Wires*—The electrical lead wire shall be of such dimension that its electrical resistance for the length used will not disturb the electrical circuit by more than 0.0001 V. This has been accomplished by using no more than a total of 500 linear ft (150 m) of at least AWG No. 24 wire. The wire shall be suitably coated with direct burial type of insulation.

4. Procedure

4.1 *Spacing Between Measurements*—While there is no pre-defined minimum spacing between measurements on the surface of the concrete member, it is of little value to take two measurements from virtually the same point. Conversely, measurements taken with very wide spacing may neither detect corrosion activity that is present or result in the appropriate accumulation of data for evaluation. The spacing shall therefore be consistent with the member being investigated and the intended end use of the measurements (Note 2).

NOTE 2—A spacing of 4 ft (1.2 m) has been found satisfactory for evaluation of bridge decks. Generally, larger spacings increase the probability that localized corrosion areas will not be detected. Measurements may be taken in either a grid or a random pattern. Spacing between measurements should generally be reduced where adjacent readings exhibit algebraic reading differences exceeding 150 mV (areas of high corrosion activity). Minimum spacing generally should provide at least a 100-mV difference between readings.

4.2 *Electrical Connection to the Steel:*

4.2.1 Make a direct electrical connection to the reinforcing steel by means of a compression-type ground clamp, or by brazing or welding a protruding rod. To ensure a low electrical resistance connection, scrape the bar or brush the wire before connecting to the reinforcing steel. In certain cases, this technique may require removal of some concrete to expose the reinforcing steel. Connect the electrical connection to the reinforcing steel to the negative terminal of the voltmeter.

4.2.2 Attachment must be made directly to the reinforcing steel except in cases where it can be documented that an exposed steel member is directly attached to the reinforcing steel. Certain members, such as expansion dams, date plates, lift works, and parapet rails may not be attached directly to the reinforcing steel and, therefore, may yield invalid readings. Electrical continuity of steel components with the reinforcing steel can be established by measuring the resistance between widely separated steel components on the deck. Where duplicate test measurements are continued over a long period of time, identical connection points should be used each time for a given measurement.

4.3 *Electrical Connection to the Half Cell*—Electrically connect one end of the lead wire to the half cell and the other end of this same lead wire to the positive terminal of the voltmeter.

4.4 *Pre-Wetting of the Concrete Surface:*

4.4.1 Under certain conditions, the concrete surface or an overlaying material, or both, must be pre-wetted by either of the two methods described in 4.4.3 or 4.4.4 with the solution described in 3.1.2 to decrease the electrical resistance of the circuit.

4.4.2 A test to determine the need for pre-wetting may be made as follows:

4.4.2.1 Place the half cell on the concrete surface and do not move.

4.4.2.2 Observe the voltmeter for one of the following conditions:

(*a*) The measured value of the half cell potential does not change or fluctuate with time.

(*b*) The measured value of the half cell potential changes or fluctuates with time.

4.4.2.3 If condition (*a*) is observed, pre-wetting the concrete surface is not necessary. However, if condition (*b*) is observed, pre-wetting is required for an amount of time such that the needle voltage reading is constant when observed for at least 5 min. If pre-wetting cannot obtain condition (*a*), either the electrical resistance of the circuit is too great to obtain valid half cell potential measurements of the steel, or stray current from a nearby direct current traction system or other fluctuating direct current, such as arc welding, is affecting the readings. In either case, the half cell method should not be used.

4.4.3 *Method A for Pre-Wetting Concrete Surfaces* — Use Method A for those conditions where a minimal amount of pre-wetting is required to obtain condition (*a*) as described in 4.4.2.2. Accomplish this by spraying or otherwise wetting either the entire concrete surface or only the points of measurement as described in 4.1 with the solution described in 3.1.3. No free surface water should remain between grid points when potential measurements are initiated.

4.4.4 *Method B for Pre-Wetting Concrete Surfaces* — In this method, saturate sponges with the solution described in 3.1.3 and place on the concrete surface at locations described in 4.1. Leave the sponges in place for the period of time necessary to obtain condition (*a*) described in 4.4.2.2. Do not remove the sponges from the concrete surface until after the half cell potential reading is made. In making the half cell potential measurements, place the electrical junction device described in 3.1.2 firmly on top of the pre-wetting sponges for the duration of the measurement.

4.5 *Underwater, Horizontal, and Vertical Measurements:*

4.5.1 Potential measurements detect corrosion activity, but not necessarily the location of corrosion activity. The precise location of corrosion activity requires knowledge of the electrical resistance of the material between the half cell and the corroding steel. While underwater measurements are possible, results regarding the location of corrosion must be interpreted very carefully. Often it is not possible to precisely locate points of underwater corrosion activity in salt water environments because potential readings along the member appear uniform. However, the magnitude of readings does serve to indicate whether or not active corrosion is occurring. Take care during all underwater measurements that the half cell does not become contaminated and that no part other than the porous tip of the copper-copper sulfate electrode half cell comes in contact with water.

4.5.2 Perform horizontal and vertically upward measurements exactly as vertically downward measurements. However, additionally ensure that the copper-copper sulfate solution in the half cell makes simultaneous electrical contact with the porous plug and the copper rod at all times.

4.6 *Care of the Half Cell* — The porous plug should be covered when not in use for long periods to ensure that it does not become dried to the point that it becomes a dielectric (upon drying, pores may become occluded with crystalline copper-sulfate). If cells do not produce the reproducibility or agreement between cells described in 3.1.1, cleaning the copper rod in the half cell may rectify the problem. The rod may be cleaned by wiping it with a dilute solution of hydrochloric acid. The copper sulfate solution should be renewed either monthly or before each use, whichever is the longer period. At no time should steel wool or any other contaminant be used to clean the copper rod or half cell tube.

5. Recording Half Cell Potential Values

5.1 Record the electrical half cell potentials to the nearest 0.01 V. By convention, a negative (−) sign is used for all readings. Report all half cell potential values in volts and correct for temperature if the half cell temperature is outside the range of 72 ± 10°F (22.2 ± 5.5°C). The temperature coefficient for the correction is given in 3.1.1.3.

6. Data Presentation

6.1 Test measurements may be presented by one or both of two methods. The first, an equipotential contour map, provides a graphical delineation of areas in the member where corrosion activity may be occurring. The second method, the cumulative frequency diagram, provides an indication of the magnitude of affected area of the concrete member.

6.1.1 *Equipotential Contour Map*—On a suitably scaled plan view of the concrete member, plot the locations of the half cell potential values of the steel in concrete and draw contours of equal potential through points of equal or interpolated equal values. The maximum contour interval shall be 0.10 V. An example is shown in Fig. 3.

6.1.2 *Cumulative Frequency Distribution*—To determine the distribution of the measured half cell potentials for the concrete member, make a plot of the data on normal probability paper in the following manner:

6.1.2.1 Arrange and consecutively number all half cell potentials by ranking from least negative potential to greatest negative potential.

6.1.2.2 Determine the plotting position of each numbered half cell potential in accordance with the following equation:

$$f_x = \frac{r}{\Sigma n + 1} \times 100$$

where:

f_x = plotting position of total observations for the observed value, %,

r = rank of individual half cell potential, and

Σn = total number of observations.

6.1.2.3 Label the ordinate of the probability paper "Half Cell Potential (Volts, CSE)," where CSE is the designation for copper-copper sulfate electrode. Label the abscissa of the probability paper "Cumulative Frequency (%)." Draw two horizontal parallel lines intersecting the −0.20 and −0.35 V values on the ordinate, respectively, across the chart.

6.1.2.4 After plotting the half cell potentials, draw a line of best fit through the value (Note 3). An example of a completed plot is shown in Fig. 4.

NOTE 3—It is not unusual to observe a break in the straight line. In these cases, the line of best fit shall be two straight lines that intersect at an angle.

7. Interpretation of Results[2]

7.1 Laboratory testing of reinforced concrete specimens indicates the following regarding the significance of the numerical value of the potentials measured. Voltages listed are referenced to the copper-copper sulfate (CSE) half cell.

7.1.1 If potentials over an area are numerically less than −0.20 V CSE, there is a greater than 90 % probability that no reinforcing steel corrosion is occurring in that area at the time of measurement.

7.1.2 If potentials over an area are in the range of −0.20 to −0.35 V CSE, corrosion activity of the reinforcing steel in that area is uncertain.

7.1.3 If potentials over an area are numerically greater than −0.35 V CSE, there is a greater than 90 % probability that reinforcing steel corrosion is occurring in that area at the time of measurement.

7.1.4 In laboratory tests where potentials were numerically greater than −0.50 V, approximately half of the specimens cracked due to corrosion activity.

7.1.5 Positive readings, if obtained, generally indicate insufficient moisture in the concrete and should not be considered valid.

8. Report

8.1 The report shall include the following:

8.1.1 Type of cell used if other than copper-copper sulfate.

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[2] The following published reports give supportive detail for interpretation of results: Spellman, D. L., and Stratfull, R. F., "Concrete Variables and Corrosion Testing," *Highway Research Record No. 433*, 1973; Stratfull, R. F., "Half-Cell Potentials and the Corrosion of Steel in Concrete," *Highway Research Record No. 423*, 1973; and Clear, K. C., and Hay, R. E., "Time to Corrosion of Reinforcing Steel in Concrete Slabs," Federal Highway Administration, Vol 1 and 2, *Interim Reports FHWA-RD-73-32 and 33*, April 1973.

8.1.2 The estimated average temperature of the half cell during the test.

8.1.3 The method for pre-wetting the concrete member and the method of attaching the voltmeter lead to the reinforcing steel.

8.1.4 An equipotential contour map, showing the location of reinforcing steel contact, or a plot of the cumulative frequency distribution of the half cell potentials, or both.

8.1.5 The percentage of the total half cell potentials that are more negative than −0.35 V.

8.1.6 The percentage of the total half cell potentials that are less negative than −0.20 V.

9. Precision

9.1 The difference between two half cell readings taken at the same location with the same cell should not exceed 10 mV when the cell is disconnected and reconnected.

9.2 The difference between two half cell readings taken at the same location with two different cells should not exceed 20 mV.

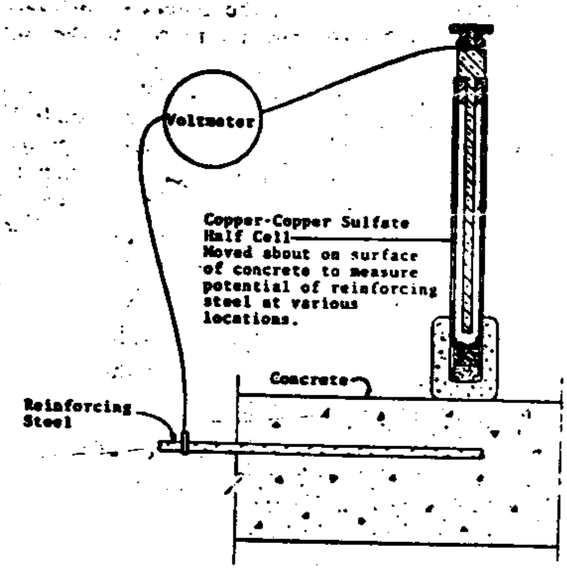

FIG. 1 Copper-Copper Sulfate Half Cell Circuitry.

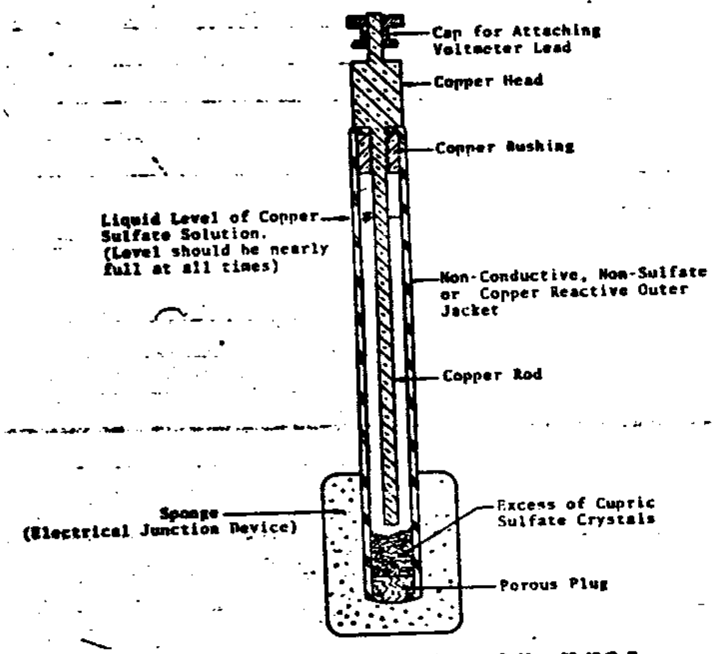

FIG. 2 Sectional View of a Copper–Copper Sulfate Half Cell.

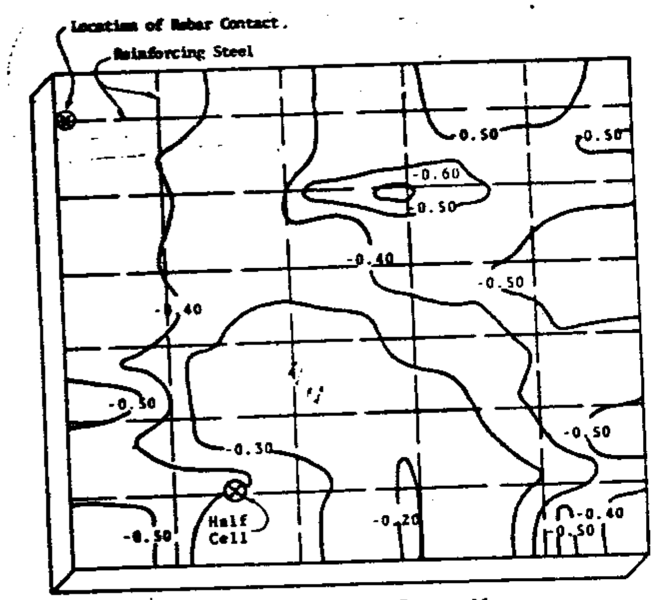

FIG. 3 Equipotential Contour Map.

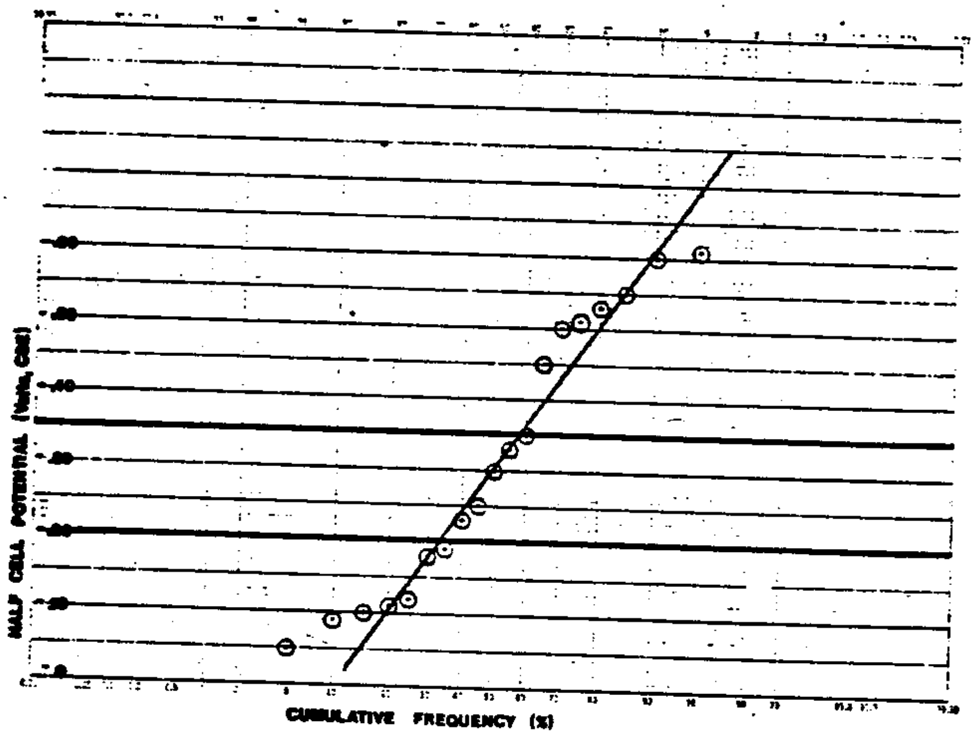

FIG. 4 Cumulative Frequency Diagram.

Standard Test Method for ELECTRICAL RESISTIVITY OF MEMBRANE-PAVEMENT SYSTEMS[1]

This Standard is issued under the fixed designation D 3633; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

1. Scope

1.1 This method covers the measurement of the electrical resistivity of membrane-pavement systems when applied to concrete bridge decks.

1.2 Measurements shall be performed on the bituminous pavement surface covering the waterproofing membrane.

1.3 This method utilizes a measure of electrical resistance between the saturated top surface of the membrane and the reinforcing steel embedded in the concrete bridge deck.

1.4 The values measured represent the electrical resistance obtained with the equipment and procedures stated herein and do not necessarily agree or correlate with those using other equipment or procedures.

NOTE 1—The values stated in SI units are to be regarded as the standard.

2. Significance and Use

2.1 The method for measuring the electrical resistivity of membrane-pavement systems may be interpreted to indicate the effectiveness of such systems.

2.2 The method is predicated on the fact that an electrical connection between the surface of the pavement and the reinforcing steel in the concrete pavement cannot be made through an impermeable insulating membrane.

2.3 The method may be used for acceptance when the accepting agency specifies the minimum resistance value desired.

3. Apparatus

3.1 *Ohmmeter*, d-c, 20 000 Ω/V rating connected to a double-pole, double-throw switch box or a-c ohmmeter (switch box not required).

NOTE 2—When this method is used for acceptance, the accepting agency should specify the type of ohmmeter to be used.

3.2 *Insulated Wire*, No. 18, Belden test probewire or equivalent. Two spools, minimum 38 m (125 ft).

3.3 *Copper Plate*, 305 by 305 by 3.0 mm (12 by 12 by 1/8 in.), with the means for connecting the ohmmeter lead and a wooden handle approximately 1 m (39 in.) in length.

3.4 *Polyurethane Sponge*, 305 by 305 by 13 mm (12 by 12 by 1/2 in.), to be attached to the copper plate with rubber bands or other suitable means. When assembled this apparatus is called the probe.

3.5 *Pressure Spray Can*, 12-litre (3-gal) capacity, with a hose and spray nozzle.

4. Reagent

4.1 A wetting agent which, when added to the water, will break the surface tension and promote the penetration of the water through the bituminous pavement.

5. Sampling

5.1 Determine locations on the bridge deck to be tested by using either a grid pattern similar to that illustrated in Fig. 1 or by a random location system that will ensure that the bridge deck area to be tested will be adequately represented.

6. Procedure

6.1 Prepare the surface to be tested by removing all foreign material by sweeping or

[1] This method is under the jurisdiction of ASTM Committee D-4 on Road and Paving Materials, and is the direct responsibility of Subcommittee D04.36 on Bridge Deck Protective Systems.

Current edition approved Nov. 25, 1977. Published January 1978.

scraping, or both. Do not use water to clean. The surface must be dry and clean before testing.

6.2 Uncoil an ample length of wire to reach all areas to be tested, attach the minus (−) jack of the ohmmeter to the reinforcing steel and the plus (+) jack to the 305 by 305 by 3.0-mm (12 by 12 by 1/8-in.) copper plate of the probe. Then saturate the sponge with water containing the wetting agent.

NOTE 3—A direct connection from the ohmmeter to the top layer of reinforcing steel in the deck is desirable. However, if it is not practical to do this, the bridge railing, expansion joints, light standards, drainage scuppers, or other exposed steel, which is known to have contact with the top mat of reinforcing steel, may be used to provide this connection.

6.2.1 Check the ohmmeter battery for satisfactory charge, then zero the ohmmeter dial indicator.

6.2.2 Check for overall equipment operation and satisfactory circuit by placing the probe on exposed concrete deck curbing and observing the resistance reading on the ohmmeter. This reading will normally range from 1000 to 3000 Ω for various bridge decks. Place the probe at several locations along the curb and observe the readings, which must remain relatively constant as an indication of a complete electrical circuit and especially to ensure a good contact with the reinforcing steel.

NOTE 4—Excessive moisture in the pavement would cause invalid resistance readings. To give a measure of assurance against this, the resistance between any two test sites may be checked prior to the testing sequence described in 6.3 through 6.4. This is accomplished by attaching the ohmmeter to two probes, rather than a single probe, and the reinforcing steel. Immediate low readings (10 000 ± Ω) will indicate excessive moisture in the pavement and on top of the membrane and further testing of the entire deck, or at least such identified portions of it, should be postponed.

6.3 By means of the pressure spray can containing water mixed with 8 ml/litre (1 oz/gal) of wetting agent, wet a spot at each test location as determined in 5.1. Each spot should be large enough to accommodate the probe. Repeat the wetting of each test site three to five times in series. Take care that the wetted areas do not interconnect, either on the surface or at the pavement-membrane interface.

6.3.1 In order to be sure that the applied water has penetrated the pavement and is in contact with the membrane, select a representative, well-compacted, test location as a check point. After allowing ample time for moisture penetration at all test sites, place the probe on the check point and determine the resistance with the test apparatus assembled as described in 6.2. Repeat the wetting of all test sites followed by a determination of the resistance at the check point. Do this until the check point resistance reading has stabilized. The time required for the wetting process will vary, depending on the thickness and permeability of the pavement layer (30 min is normally sufficient).

NOTE 5—The reasoning behind the above requirement is that when the dense, very well-compacted pavement at the check point, which has been wetted equally along with all other test locations, achieves a stabilized resistance, then logically all other test sites should have stabilized also. It has been found expedient in some cases (due to high ambient temperatures) to place prewetted sponges at each test site after wetting in order to maintain saturation.

6.4 Proceed to test the bridge deck for electrical resistance. Place the probe at each test location and allow the ohmmeter reading to settle; note it. For a d-c ohmmeter, reverse the leads by tripping the double-pole switch. When the reading has settled again, note it and record the average of two such readings as the resistance for each point. (See Fig. 2 for an example of the recording procedure.)

NOTE 6—Reversal of the d-c meter leads by means of the switch box and averaging the readings reduces the error induced by galvanic coupling of the probe and the reinforcing steel. This is not necessary when using an a-c ohmmeter. If readings are taken using scales other than the highest range provided by the ohmmeter, note the scale used.

6.5 If it is desired to further define areas of low-resistance readings, establish a grid pattern with intersections at intermediate points not previously tested. These points should only be tested after allowing sufficient time for the moisture in the pavement to dissipate. Time for this will depend on the density and thickness of the pavement, as well as the ambient and pavement temperatures (usually a minimum of 24 h should be allowed).

7. Report

7.1 The resistance values shall be reported in a manner similar to that shown in Fig. 2. Areas that fail to meet any minimum requirements may be outlined on the grid sheet. If desired, these same areas may be outlined on the bridge deck. Any repairs or corrections that need to be made shall be noted on the report sheet.

8. Precision and Accuracy

8.1 Within the scope and significance of this method, a precision or accuracy statement has not been developed. A statement may be developed at a future date when more experience is accumulated.

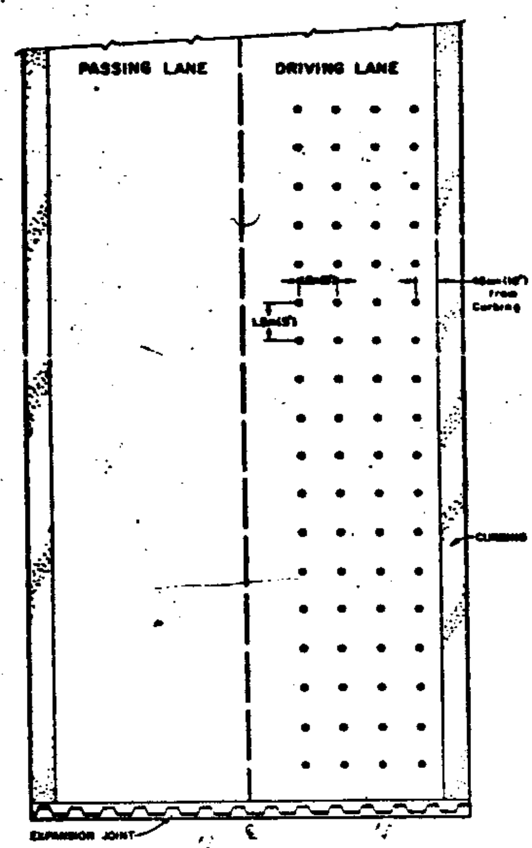

FIG. 1 Layout for Test Grid.

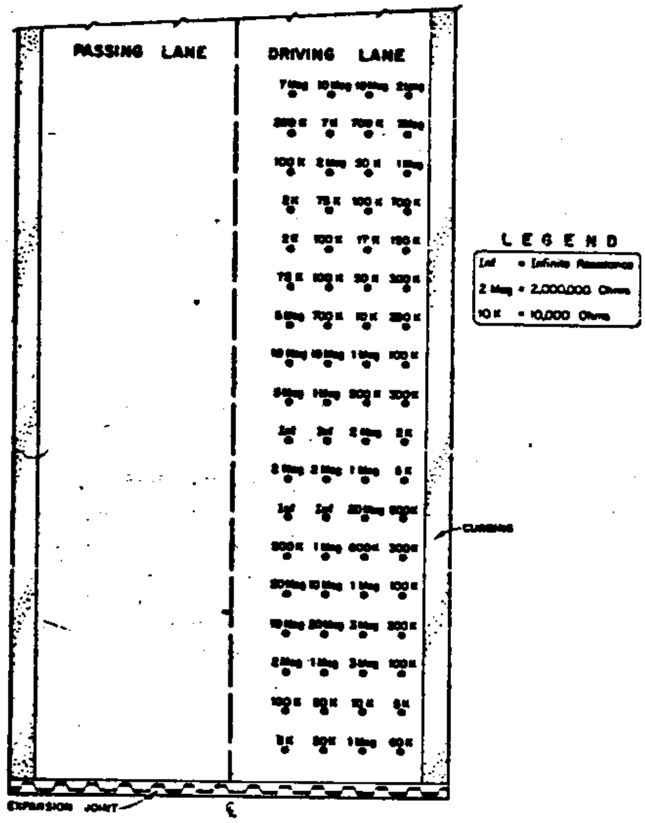

FIG. 2 Typical Resistance Values.

The American Society for Testing and Materials takes no position respecting the validity of any patent rights asserted in connection with any item mentioned in this standard. Users of this standard are expressly advised that determination of the validity of any such patent rights, and the risk of infringement of such rights, is entirely their own responsibility.

This standard is subject to revision at any time by the responsible technical committee and must be reviewed every five years and if not revised, either reapproved or withdrawn. Your comments are invited either for revision of this standard or for additional standards and should be addressed to ASTM Headquarters. Your comments will receive careful consideration at a meeting of the responsible technical committee, which you may attend. If you feel that your comments have not received a fair hearing you should make your views known to the ASTM Committee on Standards, 1916 Race St., Philadelphia, Pa 19103, which will schedule a further hearing regarding your comments. Failing satisfaction there, you may appeal to the ASTM Board of Directors.

APPENDIX G

FIELD TRIP REPORTS

FIELD TRIP REPORT

Bridge 285/10, Columbia River

On Thursday, February 23, 1983, Khossrow Babaei, TRAC research engineer, accompanied by Tom Roper, WSDOT Bridge Branch senior associate bridge engineer, with Keith Anderson, assistant special projects engineer, and Ronald Schultz, special projects engineer, both of the WSDOT Materials Laboratory, visited Bridge 285/10 in Wenatchee, Washington, 0.3 miles (0.5 km) west of Junction SR 28. Prior to visiting the site, the tentative testing plan prepared by TRAC was explained to David Malsch, District 2 materials engineer in Wenatchee.

Generally, the objective of this visit was to visually observe the deck and evaluate the location of selected test sections on the deck. During the observation, it was noticed that the spacing of the joints on the deck was about 32 ft (9.7 m) as indicated in the bridge plans. Therefore, the joint spacing indicated in the Materials Laboratory survey of 1975 (i.e., 24 ft or 7.3 m) (see Appendix D) was disregarded and the test sections were located to conform with the 32 ft (9.7 m) joint spacing as well as the number of joints and expansion dams shown in the Materials Laboratory survey

Due to adverse weather conditions and heavy traffic, complete investigation of the condition of the deck was not possible. However, it was noticed that the AC overlay was almost in good condition and there was a transverse crack on the AC for almost every joint on the deck. Nicks and spalls could also be seen along the edges of the curbing. No shoulder was located on the deck and the deck was divided into four lanes.

FIELD TRIP REPORT

Bridge 5/650 W, Ebey Slough

On Thursday, March 3, 1983, Khossrow Babaei, TRAC research engineer, accompanied by Tom Roper, WSDOT Bridge Branch senior assistant bridge engineer, and Keith Anderson, WSDOT Materials Laboratory assistant special projects engineer, visited Bridge 5/650 W, 4.6 miles (7.7 km) north of Junction SR 2 in Washington.

The objective of this visit was to visually observe the deck and evaluate the location of the selected test section on the deck. During the observation, it was noticed that the test section was located on a patched and repaired AC and maybe a distorted membrane. This severe deterioration could be related to the high concentration of the chlorides as well as the high grade on the deck (i.e., about 2.24%) at this location. Therefore, a new test section was located with no patches on the AC. The grade on the new test section is zero and the chloride concentration, according to the Materials Laboratory survey of 1974, is about 2.31 lb/cy (1.36 kg/m), which is the highest on the deck.

Further observation of the deck showed that, aside from the patched area located at the south end of the bridge and on the grade, the AC was in fair condition with some ravelling in the wheel lines. Transverse cracks on the AC and adjacent to almost every expansion dam were also noticed.

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II. PART TWO

FIELD TESTING AND DATA COLLECTION

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1. INTRODUCTION

The first part of the study entitled "Performance Evaluation of Waterproofing Membrane Protective Systems for Concrete Bridge Decks" was generally focused on obtaining background information regarding the conditions of three test bridges in Washington as well as designing a detailed testing plan in order to evaluate the performance of the waterproofing membranes applied on them. The following is a list of the test bridges and their locations:

1. Bridge 285/10 (Columbia River) near Wenatchee
2. Bridge 82/106 (Roza Canal) near Yakima
3. Bridge 5/650 W (Ebey Slough) near Everett

This part of the study (Part Two) involves field testing and data collection from the above bridges in accordance with the testing plan prepared in Part One and in conjunction with the WSDOT Materials Laboratory.

Stated briefly, the field testing program included the following experiments which were conducted by the WSDOT Materials Laboratory with the assistance of the WSDOT Bridge Branch and TRAC personnel:

1. Electrical resistivity testing
2. Chain dragging
3. Electrical potential testing
4. Depth of rebar measurement (pachometer survey)
5. Core sampling AC overlay and membrane
6. Chloride content measurement

In addition, as a requirement for testing Bridge 285/10, a portion of the AC overlay and membrane on the deck was removed and after testing was replaced. This portion of the work was done by the District 2 maintenance crew.

Complying with the testing schedule, the actual date of testing for each bridge was coordinated with the weather. Field testing of Bridges 5/650 and 82/106 was accomplished during the month of April in accordance with the initial testing plan. However, the testing of Bridge 285/10, which required overlay and membrane removal and replacement, was revised due to the results of a field trial conducted in advance of the testing. During the field trial, a small portion of the asphalt overlay and membrane was removed and the procedure was evaluated. The results from this trial indicated that it would be very time-consuming to remove the pavement and clean the concrete surface as initially planned. Thus, a revised testing plan was prepared for Bridge 285/10 which minimized pavement removal and disruption to traffic as well. A description of the field trial and revised testing plan are included in Section 2. The actual testing of Bridge 285/10 was accomplished during the month of May.

Regardless of some minor changes in the work plan due to actual field conditions, the testing phase generally followed the experimental design in every aspect of testing and no major difficulties were encountered in the field. After testing the first bridge (Eby Slough), the test team were able to coordinate their activities very efficiently. This maximized the rate of obtaining data and minimized traffic problems. A description of the different experimentations and field activities conducted during the testing phase can be found in Section 3.

After the raw data, including both field and laboratory data, were collected, they were tabulated, analyzed and plotted with a suitable format so that a reasonable assessment of the conditions of the membranes as well as the concrete decks could be made. Section 4 of this report is allocated for this purpose. Included also in Section 4 are the "before" condition data (if available) arranged in the same format to make comparison easier. Although in some cases the lack of "before" condition data may make comparison impossible, the new data could be used as baseline information in the future to assess the service lives of the waterproofing systems as well as the rate of deterioration of the decks.

2. PAVEMENT REMOVAL FIELD TRIAL AND REVISED TESTING PLAN FOR COLUMBIA RIVER BRIGE (285/10)

The initial testing plan of Bridge 285/10, as reported in Part One of this study, required removing the existing asphalt concrete (AC) overlay and waterproofing membrane on three test sections, each 96 ft (29.1 m) long by 11.5 ft (3.5 m) wide located on the outside lane southbound, with a total area of 3312 sq ft (304.1 sq m) (see Figure 1 for layout of the test sections). The AC overlay is a class B mix and the waterproofing membrane is a hot-applied rubberized asphalt. The plan called for evaluation of the pavement removal operation in advance of actually testing the bridge. On March 24, 1983 a field trial was conducted on Bridge 285/10 to remove a small portion of the AC overlay and membrane (i.e., 12 ft or 3.6 m long by 10 ft or 3.0 m wide, as shown in Figure 1) and to evaluate the procedure.

The results of this trial indicated that it would be very time-consuming to completely remove the pavement and clean the concrete surface at all three test sections. Thus, a revised testing plan was prepared for Bridge 285/10 to minimize pavement removal and disruption to traffic while still obtaining the essential data to evaluate the condition of the membrane.

2.1. Description of Field Trial

Prior to pavement removal, the AC overlay in the trial area was chain dragged. Chain dragging did not detect any defects under the AC overlay. Next, an air hammer was used to cut the perimeter of the trial area up to the surface of the concrete. Vibration caused by the hammer broke the bond between the membrane and concrete in those areas adjacent to the cut, as was indicated by subsequent chain dragging. A motor grader was then used to scarify the AC overlay and membrane longitudinally (i.e., parallel to the deck centerline) to divide the area into more than one piece. However, this resulted in scarifying the concrete surface in some locations as well. Subsequently, a front-end loader was used to lift the AC overlay up by starting from the cut perimeter and moving in a direction parallel to the deck centerline. Pavement was lifted up together with the membrane in the areas with broken bond (i.e., in the vicinity of the cut). Nevertheless, the pavement could not be lifted easily in other areas due to a strong bond at the interface. The hammer was used once again to break the bond by vibration in a few locations within the trial area. This eased removing the pavement and membrane, but only in the vibrated areas.

After the AC overlay and membrane were removed completely, a dark colored stain from the membrane which did not contain any rubber particles remained on the concrete surface. In spite of the dark color, the concrete surface could be readily inspected. There was no evidence of moisture between the concrete surface and membrane, although it had rained two days before the field trial. Some transverse cracks could be seen on the concrete deck. A chain dragging conducted on the exposed concrete indicated no delaminations in the concrete. Subsequently, to visually inspect the concrete surface more clearly and to clean the surface completely, it was sand blasted. Although somewhat time-consuming, sand blasting was successful in totally cleaning and exposing the concrete surface.

The entire pavement removal operation, including the time spent on adjusting some of the equipment, took about three hours.

2.2. Detailed Revised Testing Plan

Three test sections, each 96 ft (29.1 m) long by 11.5 ft (3.5 m) wide, were located on the outside lane southbound, with a total testing area of 3312 sq ft (304.1 sq. m) as shown in Figure 2. The locations of two of the test sections (i.e., Nos. 1 and 3) were the same as "old" test sections Nos. 1 and 2, respectively (see Figures 1 and 2). However, the location of the other test section (i.e., No. 2) did not coincide with "old" test section No. 3, since there was no evidence of breakdown in the membrane or deterioration of the concrete in the proximity of "old" test section No. 3 during the pavement removal trial.

The concrete of test section Nos. 1 and 2 in the revised testing plan had been heavily repaired by excavating the delaminations and patching prior to installation of the membrane, whereas the concrete of test section No. 3 did not have any delaminations and repairs before waterproofing. Each test section included an area of pavement removal which is 32 ft (8.7 m) long by 8.5 ft (2.6 m) wide (see Figures 2 and 3). The pavement removal areas were located on the portion of the test sections which had the highest degree of delamination and repair in advance of waterproofing. Exposing these areas and examining them would reveal the effectiveness of the repair method as well as the protective system.

2.2.1. Sequence of Testing Activities

The testing activities on each test section on Bridge 285/10 and their sequence were revised as follows (see also Figure 5 for a summary of testing activities):

(A) Photograph and map the AC overlay surface condition including cracks of all types, raveling, patching, shoving, wet spots, etc. Photograph and record the condition of the curbs, such as cracks and spalling. Photograph and record the conditions of the deck from below, such as cracks, seepage, efflorescence, wet spots, spalling, etc.

(B) Chain drag the AC overlay to locate possible debonding between the concrete slab and AC overlay. Map and outline the areas of debonding.

(C) Conduct electrical resistivity testing (ASTM D3633) on AC overlay using the grid pattern shown in Figure 3 and record the resistivity of the membrane at the intersecting points of the grid. This pattern is adapted from

ASTM D3633, which requires 5 ft (1.5 m) spacing between data points and at least 1.5 ft (0.5 m) clearance from curbing.

(D) Obtain a minimum of 8 core samples through AC and membrane by dry sawing at the intersecting points of the grid pattern used for electrical resistivity testing. The location of core samples should be randomly selected and spaced out over the portion of the test section not to be exposed. Examine the core samples, and characterize and record the condition of the membrane using the data form shown in Figure 4.

(E) Conduct electrical potential testing (ASTM C876) in the core locations where the membrane has been punctured or removed and record the temperature. The number of readings will be equal to the number of holes drilled. In addition, record the depth of rebar at the highest and lowest half-cell readings using a pachometer.

(F) Waterproof core hole locations with a selected material and patch. (This activity could be conducted after Item M below).

(G) Remove the existing AC overlay and membrane from the designated area on the test section (see Figure 2) and sand blast the concrete surface to clean the surface sufficiently so that cracks and other distress of the concrete will be exposed.

(H) Photograph and map the exposed concrete surface condition, including all cracks, spalling, patching, scaling, popouts, exposed rebar, and any other defects noted.

(I) Chain drag the entire exposed concrete surface, noting all apparent delaminations. Map and outline the areas of delamination.

(J) Conduct a pachometer survey on the exposed concrete and record the depth of rebar at three points on the delaminated surface and three points on the sound concrete.

(K) Conduct electrical potential testing (ASTM C876) on the exposed section using the electrical potential testing grid pattern shown in Figure 3. The intent is to collect half-cell data comparable to the resistivity data if possible. Additional measurements between the grid points should be taken in the areas of high corrosion activity where adjacent readings exhibit differences exceeding 0.15 v.

(L) Determine chloride content at two locations, the highest and lowest half-cell readings, at the level of rebar (this could include the core locations as well).

(M) Determine chloride content in the vicinity of each previous chloride content data point taken in 1975 on the entire deck and at the same depth (i.e., a total of four points on the entire deck).

(N) Apply a new membrane on the exposed area.

(O) Upon completion of the membrane application, but prior to repaving AC overlay, take electrical resistivity readings as described in Item C above.

(P) Repave the test section with asphalt concrete and photograph and record any damage done to the membrane during this period.

(Q) After completion of the overlay, chain dragging should be repeated once again as explained in Item B above to check the integrity of the new pavement and concrete deck for possible debonding.

(R) Following chain dragging, take electrical resistivity readings once again on the new AC overlay to determine if any damage to the membrane has occurred during construction of the overlay.

3. FIELD TESTING PROGRAM

The different experiments conducted in the testing phase of this study could be categorized as follows:

A. Experiments conducted routinely by WSDOT for inspection of bridge decks.

- A-1. Visual inspection of AC overlay and/or concrete deck to detect surface distress.
- A-2. Chain dragging AC overlay and/or concrete deck to detect defects under AC overlay and/or delaminations in the concrete.
- A-3. Chloride analysis of concrete samples obtained at the decks to determine the chloride content at any depth in the concrete.
- A-4. Electrical resistivity testing of membrane to evaluate the permeability of membrane under service. (This test is not used by WSDOT for routine inspection of membrane waterproofed decks under service. However, it is specified to evaluate waterproofing effectiveness of newly installed membrane/pavement systems).

B. Experiments not conducted routinely by WSDOT for inspection of bridge decks.

- B-1. Core sampling AC and membrane to characterize the conditions of the membrane and the concrete underneath as well. Also, to be able to conduct electrical potential testing on the exposed concrete in the core location.
- B-2. Electrical potential testing to detect corrosion of top mat reinforcing steel.
- B-3. Pachometer survey to determine the depth and/or location of the rebar in concrete deck.

In addition to the above experiments, the field activities included rebar exposure at each test bridge to provide ground for electrical resistivity and potential testing and pavement/membrane removal and replacement at Bridge 285/10 to investigate the conditions of the exposed concrete deck.

3.1. Visual Inspection

Unlike exposed concrete decks, visual inspection of asphalt covered decks is difficult and needs more experience. Since the deck is overlayed, the concrete defects might be undetected at the time of inspection. During the visual inspection of the asphalt-covered test bridges, there was no major clue which would denote the conditions of the deck slabs except for a portion of Bridge 5/650 W, which was patched, possibly due to distress of underneath concrete (see Figure 6). The condition of AC overlay on the test bridges ranged from fair (Bridges 5/650 W and 285/10) to good (Bridge 82/106 L/S) and no cracks could be detected on the test sites. Nevertheless, there were some transverse cracks along the expansion dams on Bridge 5/560 W (see Figure 7) and directly above the slab joints covered with asphalt overlay on Bridge 285/10 (see Figure 8) which might have been caused by other sources rather than chloride-induced corrosion. The conditions of the curbs ranged from fair (Bridge 5/650 W) to good (Bridges 82/106 L/S and 285/10) with no noticeable cracks or spalling at the test sites. However, the curbs were scraped on the edge (Bridges 5/650 W and 285/10), possibly due to traffic abrasions (see Figure 9). A thorough examination of the decks from the underside was impossible since the undersides of the decks were nearly inaccessible. However, efflorescence could be seen intensively on the underside of Bridge 5/560 W and 285/10 (see Figure 9-1) and rarely on the underside of Bridge 82/106 LS (see Figure 9-2). In general, except for a portion of Bridge 5/650 W on grade which was patched with asphalt (see Figure 6), no major key pointers to defective concrete beneath the AC overlay could be found.

Visual inspection of exposed concrete was conducted only on Bridge 285/10. After the exposed concrete was sand blasted, the previously repaired areas on the original concrete could be clearly distinguished (see Figure 10). Sand blasting also removed the loose concrete from the deck, thus providing more opportunity to detect the distressed areas such as spalls and exposed rebars (see Figure 11).

3.2. Chain Dragging

The device used for chain dragging was a simple shopmade tool. A 1/2 in (13 mm) diameter steel pipe was fabricated into a "T" shape, with wheels at the tips of the flanges. Fastened to the flange of the "T" were six 3/8 in (10 mm) steel chains spaced uniformly (see Figure 12). Pushing the device along the deck resulted in a dull sound to be heard when delaminated areas in the concrete or defects under AC overlay were encountered. The boundaries of the defective areas were defined with just two or more passes of a single chain (see Figure 13).

In general, chain dragging the AC overlays could not completely define the poor bond under AC overlay and in the interface as identified in the core locations. Neither was the procedure capable of determining the delaminations in the concrete as defined by a subsequent pavement removal at test section 1 of Bridge 285/10. However, it detected defects under AC overlay on test section 2 of Bridge 285/10. The results of a subsequent visual inspection and chain dragging the exposed concrete on a portion of test section 2 suggests that the defects detected by chain dragging AC overlay may mainly be highly progressed delaminations and/or spalls in the concrete deck (see Sections 4.1.2 and 4.2.2). At the Bridge 5/650 W test site, chain dragging the AC overlay detected defects under AC overlay and along the expansion dams (see Figure 7). In this case also, the defects may be extensive delaminations or spalling in the concrete produced by pounding of traffic loads. Recently the results of research conducted by the Ontario Ministry of Transportation [13] have shown that by chain dragging asphalt covered decks, areas of debonding between AC overlay and concrete deck cannot be detected. But the same study has indicated that the procedure is capable of identifying surface concrete scales directly under AC overlay and about 13% of the total delaminations in the concrete.

3.3. Sampling Concrete for Chloride Analysis

Concrete sampling for chloride content measurement was done by a rotary percussion hammer with a 1 in (25 mm) diameter carbide-tipped precision bit which obtained powdered concrete samples from the concrete deck for laboratory wet chemical analysis (see Figures 14, 14-1 and 14-2). Concrete sampling was mainly done either in the AC and membrane core locations (see Section 3.5) or where the overlay was removed, eliminating drilling through AC overlay. The samples were taken by drilling a hole to a depth of about 0.5 in (13 mm) above the level of the desired depth and removing the powdered concrete with a vacuum cleaner before drilling the additional 0.5 in (13 mm). The drilling depth was checked periodically during the drilling process by inserting a scale into the hole until the proper depth was reached. An alternative to this procedure may be the use of a scribed depth indicator attached to the rotary hammer to insure the proper sampling depth is reached while drilling (for more information see [14]).

The use of a rotary percussion hammer for the concrete sampling has the advantage of speed and economy. Since the concrete samples were not pulverized under controlled laboratory conditions, extra care was taken not to contaminate samples at the sampling depth by abrading concrete from the sides of the hole. An alternative to overcome this problem, as reported in [7], may be the reduction of the core diameter as each successive sample is taken.

3.4. Electrical Resistivity Testing

Electrical resistivity of waterproofing membranes is used as a measure of their impermeability. The test was conducted after visual inspection and chain dragging the AC overlays (ASTM Test Method D3633). The resistivity measuring device (probe) was a 12 in (30.5 cm) by 12 in (30.5 cm) copper plate with an electrical connection made to the plate. One sponge, 12 in (30.5 cm) by 12 in (30.5 cm), was fastened to the bottom of the plate by means of two rubber bands in order to facilitate contact with the deck surface. A handle was attached to the plate for convenience in moving and placing the probe at various locations on the deck (see Figure 15). Prior to testing, the grid points on the AC overlay were located and wetted by means of a pressure spray can containing water mixed with a wetting agent (see Figure 16). Each wet spot was large enough to accommodate the probe. The solution was also used to saturate the sponge attached to the probe. A digital DC ohmmeter was used to record the resistivity of the membranes. The plus (+) lead of the ohmmeter was attached to the probe and the minus (-) lead was attached directly to the top mat reinforcing steel using a compression type ground clamp (see Figure 17). The latter was accomplished by locating a rebar on the deck by means of a pachometer and coring until it was exposed (see Section 3.8). Meanwhile, the wetting of the test sites was repeated to keep the solution penetrating into the AC overlay. The probe was then placed at the first test location and resistivity was recorded. Resistivity was measured at a few succeeding test locations as well. Following this, the resistivity measurement was repeated in the same sequence. When the measurements stabilized and they were constant at those locations, the soaking time was considered sufficient and the readings were recorded at all grid points on the test sections.

High resistance measured between the exposed rebars (ground) and other elements such as expansion dams and bridge railings on Bridges 5/650 W and 82/106 indicated that there was no electrical continuity between these elements and the top mat reinforcing steel, whereas in one case (Bridge 5/650) the resistance between two exposed rebars (two grounds) 50 ft (15 m) apart on the test section was only 2 ohms, which is an indication of electrical continuity within the top mat reinforcing steel. An additional check to insure a good electrical continuity within the top mat reinforcing steel was accomplished by placing the probe at several locations along the exposed concrete curbing. In this case, the resistivity was always in the range of 1000 to 2000 ohms (i.e., the range of resistivity of exposed concrete), which is an indication of a satisfactory circuit.

3.5. Coring Program

Subsequent to electrical resistivity testing, core samples of AC overlay and membrane were taken at the grid points of resistivity testing, selected randomly on the test sections, in order to clarify the following:

A. Conditions of the membrane/pavement system

- A-1. Bond of membrane to AC overlay and concrete deck
- A-2. Defects in membrane
- A-3. Presence of moisture under membrane

B. Condition of exposed concrete

C. State of rebar corrosion by means of a half-cell

In addition, concrete samples for chloride analysis were taken directly in core locations, eliminating drilling through AC overlay.

A truck-mounted, nitrogen-cooled coring rig was used to core sample the AC overlays down to the surface of membrane (see Figure 18). The core diameter was 4 in (10.2 cm) and the coring process took about 5 minutes per core. After coring, a chisel was placed in the circumferential core cut and struck with a hammer. If the AC disbonded within itself or could not be removed easily by hand and chisel, the membrane bond to AC overlay was considered "good." Otherwise the bond was considered "fair" or "poor" depending on the examiners' opinion. Following removal of the AC overlay, the top of the membrane was checked for the presence of moisture. Next, the chisel was placed on the membrane perimeter and struck by the hammer until the membrane was cut around the perimeter. At this stage, if the membrane could be removed by hand easily the membrane bond to concrete was considered "poor." Otherwise, the bond was considered "fair" to "good" depending on the examiners' opinion. After removing the membrane the presence of moisture on the exposed concrete was checked immediately. The core samples were then taken to the WSDOT Materials Laboratory for further examination.

The above description of bond applies mainly to those membranes with fabric (Bridge 5/650 W and 82/106). In case of hot-applied rubberized asphalt membrane (Bridge 285/10), the membrane was always an inseparable part of the AC overlay (see Figure 19). Thus, the bond between the membrane and AC overlay was recorded "good" for all core samples. This situation also made the laboratory examination of the membrane difficult.

After removing the AC overlay and membrane core samples, the exposed concrete in the core locations were inspected visually for surface distress. The concrete surface in the core location was also tapped by a hammer for possible delaminations. Finally, half-cell testing (see Section 3.6) was conducted in the core location as well as concrete sampling for chloride measurement if needed (see Section 3.3).

After the testing in core locations was completed, they were waterproofed and patched with a mixture of cold asphalt (see Figures 19-1 and 19-2).

3.6. Electrical Potential Testing

Electrical potential testing was conducted using a copper-copper sulfate half-cell (ASTM Test Method C 876). The instrument was a 3 ft (76 cm) long piece of 1.5 in (3.8 cm) diameter plastic pipe with a sponge tip at the lower end used as an electrical junction device. The pipe was filled with a copper sulfate solution and a copper rod was inserted through a rubber top plug down to the sponge to complete the "saturated copper/copper sulfate half-cell."

A digital DC voltmeter was used to record the electrical half-cell potential of the reinforcing steel. One terminal of the digital voltmeter was attached to the half-cell and the other terminal directly to the top mat reinforcing steel using a compression type ground clamp (see Figure 17). The ground was the same as the one used for electrical resistivity testing. Half-cell potentials were recorded in the core locations by placing the sponge tip on the exposed concrete (see Figure 20). Where the pavement and membrane were stripped (Bridge 285/10), half-cell measurements were taken in a grid pattern on the exposed concrete (see Figure 20-1). On Bridges 5/650 W and 82/106, the conditions of the decks were such that they did not require prewetting (i.e., half-cell values did not fluctuate with time). However, on Bridge 285/10 some test locations were prewetted in order to stabilize half-cell readings.

Half-cell readings should be corrected for temperature if the half-cell temperature is outside the range of 72 $\pm$ 10°F (22 $\pm$ 5.5°C). Since the testing temperature was always within the abovementioned range, no correction was necessary. In one case (Bridge 5/650, patched area on AC overlay), one half-cell reading was taken directly over an AC patch, since it was known that no membrane (dielectric material) had been installed under the patch.

3.7. Pachometer Survey

A hand-held pachometer (James Instruments, Inc., Model C4956) was used for determination of rebar depth and location (see Figure 21). The instrument has a battery, a probe, and a scale. When the probe is placed on the deck and its long axis oriented parallel to the reinforcing steel, a distortion is recorded on the scale which is directly related to the distance between the probe and rebar. The scale of the instrument is thus calibrated to record the distance between the probe and rebar. Since the amount of the distortion is also related to the bar size, the calibration is made for different bar diameters. Prior to testing, the orientation of uppermost reinforcing steel and their size had been determined by examining the plans for the test bridges.

In this study, the pachometer was used for two reasons: first, to locate a rebar in the deck in order to expose the rebar and provide a ground for electrical resistivity and potential testing; and second, to determine the depth of rebar in order to obtain concrete samples for chloride analysis at the level of rebar. In addition, on Bridge 285/10 a pachometer survey was conducted on the exposed concrete deck in an attempt to ascertain if the presence or absence of the concrete deterioration is related to rebar depth.

In pachometer surveys, the measured values may be less than the actual depth of rebar depending on the existence of particles of magnetic materials in the concrete or asphalt overlay if the survey is conducted on an asphalt-covered deck. To compensate for this possible error, prior to coring and exposing a rebar for ground, the rebar depth was recorded by the pachometer. The actual rebar depth was then obtained after the rebar exposure using a scale. By comparing the recorded and actual values, a correction factor was established for each case.

3.8. Rebar Exposure to Provide Ground

A requirement for conducting electrical resistivity and potential testing is usually direct electrical connection (ground) to the top mat reinforcing steel. This was accomplished by coring and exposing rebar at each test section. The number of grounds on each bridge deck depended on the existence of expansion dams as well as the distance between different test sections. The procedure to provide ground was first using a pachometer to locate a rebar (see Figure 21) and then coring and removing the AC overlay and membrane as explained in Section 3.5 (see Figure 18). Subsequently, wet coring was employed to core the concrete to about the level of rebar. Next, the concrete in the core location was moved by means of a rotary percussion hammer until the rebar was reached (see Figure 22). Usually at this stage the actual rebar depth was measured and compared to the rebar depth obtained directly by pachometer to establish a correction factor for pachometer readings. The electrical connection to the reinforcing steel was made by means of a compression-type ground clamp (see Figure 17).

3.9. Pavement/Membrane Removal and Replacement

As explained in Section 2.2.1, the testing plan of Bridge 285/10 required removal and replacement of a portion of the pavement/membrane system on the deck. This was accomplished by the District 2 maintenance crew. After a pavement removal area was located on the deck, it was cut around the perimeter by a jackhammer. The jackhammer was then used to remove the pavement within the area designated for pavement removal (see Figure 23). The membrane and the AC overlay came out together leaving a dark colored stain on the surface of the exposed concrete. Subsequently, sand blasting was used to clean the concrete surface to expose repairs as well as any distress on the surface.

Sand blasting test section 1 is shown in Figure 24 and the completed job is illustrated in Figure 10.

After testing the exposed concrete and patching the spalled areas (see Figure 25), a thin layer of glue (primer) was applied to the concrete surface and the perimeter of the exposed area was sealed with an elastomeric mastic (see Figure 25-1). Next, a factory-made, rubber asphalt impregnated membrane (system A) was unrolled by hand onto the slab (see Figure 26). At the same time the membrane was flattened with a shovel to remove air bubbles and wrinkles (see Figure 27). After the membrane was applied, a hand roller was used to bring the membrane completely in contact with the deck surface (see Figure 28). After the newly installed membrane was tested for resistivity (see Figure 29), the test section was paved with asphalt concrete. The asphalt concrete mix was dumped on the test section and leveled by a motor grader, avoiding contact of the tires with the membrane. A steel-drum compactor then compacted the mix (see Figure 29-1).

4. PRESENTATION OF COLLECTED DATA

After the raw data, including both field and laboratory data, were collected, they were tabulated, analyzed, and plotted with a suitable format so that a reasonable assessment of the conditions of the membranes as well as the concrete decks could be made. Included in this section are the "before" condition data (if available) arranged in the same format to make comparison possible. Although in some cases the lack of "before" condition data may make comparison impossible, the data collected in this work could be used as baseline information in the future to assess the service lives of the waterproofing systems as well as the rate of deterioration of the decks.

4.1. Columbia River Bridge (285/10), Testing Asphalt Covered Deck

Bridge 285/10 was built in 1950. Due to extensive deterioration of the deck concrete, the deck was repaired by removing the delaminations and placing a grout on the excavated areas in 1976. Jet Set Super X grout was used to repair approximately 50% of the deteriorated areas. The remainder of the areas were patched with a high early strength 8 sack mixture of portland cement grout [1]. After completion of the repair job in 1976, the deck was waterproofed with a hot-applied reclaimed rubber asphalt membrane and a layer of class B asphalt concrete mix. More background information regarding Bridge 285/10 can be found in Part One of this study.

Testing of the asphalt covered deck was conducted on May 9 and 10, 1983. Weather conditions were clear with temperature at 70°F (21° C). Three test sections, each 96 ft (29.1 m) long by 11.5 ft (3.5 m) wide, were located on the outside lane southbound (see Figure 2). The different test reading setups on the three test sections are presented in Figures 30, 31 and 32 for test sections 1, 2 and 3, respectively. The concrete deck of test sections 1 and 2 was heavily repaired prior to the installation of the membrane in 1975, whereas the concrete deck of test section 3 did not have any delaminations and repairs prior to waterproofing.

4.1.1. Visual Inspection

The asphalt overlay on the test sections was in fair condition, showing some wear in wheel lines. No cracks could be visually detected on the pavement at the test sections. The condition of the curb was good with no noticeable deterioration. The curb, however, was scraped on the edge, possibly due to traffic abrasion (see Figure 9). A portion of the deck in the vicinity of test section 1 was inspected from the underside and considerable transverse efflorescence could be seen.

4.1.2. Chain Dragging

No defects under the pavement at test sections 1 and 3 could be detected by chain dragging the asphalt overlay. However, on test section 2, the impact of the chain on the pavement in some areas resulted in a dull sound being heard. These areas were marked and are presented in Figure 33. As an approximate estimate, they account for about 18% of the surveyed area of the outside lane limited to the test section (i.e., 96 ft or 29.1 m by 13.5 ft or 4.1 m). The percentage of defects on the area designated for pavement removal (i.e., 32 ft or 9.7 m by 8.5 ft or 2.6 m) is approximately 9%.

4.1.3. Electrical Resistivity Testing

The results of the electrical resistivity testing as recorded in the field are given in Figures 34, 35 and 36 for test sections 1, 2 and 3, respectively. The distribution curves of the resistivity readings are given in Figure 37. As shown in Figure 37, 18%, 39% and 42% of the readings on test sections 1, 2 and 3, respectively, are lower than 250,000 ohms. WSDOT specifications for newly installed waterproofing membranes suggests that no more than 30% of the readings be smaller than 250,000 ohms. It should be mentioned that on test section 3, almost all of the readings in the vicinity of the curb (i.e., row C, 1.5 ft or 0.5 m from the curb) were small (see Figure 36). This may be the result of a short circuit due to the wetness of the area along the curb, which might have been caused by the water cooling rig while coring the concrete in that area to expose rebar. Figure 37 also presents a distribution of the resistivity readings on test section 3 assuming that the readings along the curb (row C) are not valid. In this case, only about 13 % of the readings are lower than 250,000 ohms. Another point to note from Figure 37 is that about 11%, 23%, and 33% of the resistivity readings on test sections 1, 2 and 3 are smaller than 100,000 ohms, which is considered as the lower limit for a satisfactory waterproofing membrane. However, on test section 3, by ignoring the readings close to the curb, only 3% of the readings are lower than 100,000 ohms.

4.1.4. Coring Program

Location of the core samples of AC and membrane on test sections 1, 2 and 3 are shown in Figures 30, 31 and 32, respectively. As indicated in the figures, the coring program does not include the overlay removal areas on the test sections. The tabulated data from the coring program can be found in Tables 1, 2 and 3 for test sections 1, 2 and 3, respectively.

<u>Membrane</u>: The average AC and membrane thickness of the core samples actually measured at the site was about 1.8 in (4.6 cm), 1.5 in (3.8 cm), and 1.7 in (4.3 cm) for test sections 1, 2 and 3, respectively. There was integ-

rity between the membrane and asphalt overlay in all core samples and the membrane was an inseparable part of the overlay. Thus, the bond of the membrane to AC overlay was always recorded "good." Bond of membrane to concrete on test section 1 was recorded 56% of the time as "good," 33% as "fair," and 11% as "poor." On test section 2, the bond was "fair" 56% of the time and "poor" 44%. For test section 3, membrane-concrete bond was "good" 38% of the time, "fair" 38% and "poor" 25%. Because of the integrity between the membrane and AC overlay core samples, pinholes were not visually detectible.

Concrete Deck: Concrete surface in the core locations immediately after removing the membrane/AC core samples was slightly wet only 7% of the time (2 out of 27 core locations, one at test section 1 and one at test section 2). The exposed concrete in the core locations did not show any surface distress. No delaminations in the concrete could be detected by tapping the exposed concrete in the core locations with a hammer, including the areas marked as defects under the pavement and detected by chain dragging AC overlay. The average concrete cover thickness as measured by the pachometer over the AC overlay and in the vicinity of the core locations (considering the correction for magnetic bias as given in Tables 1, 2 and 3) was about 1.7 in (4.3 cm), 1.5 in (3.8 cm) and 2.0 in (5.1 cm) for test sections 1, 2 and 3, respectively. It is reported [15] that in many cases magnetic bias is at the most 0.25 in (0.6 cm) when measuring rebars to a depth of 3 in (7.6 cm) in the concrete. It is shown in Section 4.2.5 that the pachometer survey of the exposed concrete of the deck gave a magnetic bias correction of 0.2 in (0.5 cm) for a depth of 2.2 in (5.6 cm) in the concrete. However, as given in Tables 1, 2 and 3, the average correction for magnetic bias when measuring rebar depth over asphalt concrete is about 0.9 in (2.3 cm) for an average depth of 3.6 in (9.1 cm) (1.7 in or 4.3 cm AC and 1.9 in or 4.8 cm PCC). Two factors might have influenced the amount of the error when measuring the depth over the asphalt concrete. The first factor is the actual total depth (AC + PCC), which is higher than 3 in (7.6 cm). The second factor is the nature of the asphalt concrete mix itself, which might have contributed to this disparity. Almost similar errors were encountered when measuring the rebar depth on the asphalt covered decks at the Ebey Slough and Roza Canal test sites (see Sections 4.5.4 and 4.4.4).

Electrical Potential Testing: The results of electrical potential testing in the core locations as recorded in the field are given in Tables 1, 2 and 3 for test sections 1, 2 and 3, respectively. The results of half-cell readings collected in the core locations, together with those obtained on the exposed concrete at the pavement removal areas, are discussed in Section 4.2.3.

4.1.5. Chloride Content Measurement (see also Section 4.2.4)

Sampling concrete for chloride analysis where the deck was covered with asphalt concrete was limited to two locations, where the 1975 samples were taken. Although there were two more 1975 chloride data points on the deck, the original concrete on those areas was repaired and patched with a new grout in 1976, making resampling impossible. The results of the chloride content measurements, as well as the location of the data points, can be found in Table 4. As shown in Table 4, chloride has increased in the locations sampled for chloride since 1975 with an average amount of 1.46 lb/cy (0.86 kg/m^3) or 64% (average is based on 2 measurements at a depth of 1 in or 2.5 cm to 1.5 in or 3.8 cm in the concrete).

4.2. Columbia River Bridge (285/10), Testing Exposed Concrete Deck

The location of pavement removal areas and exposed concrete on test sections 1, 2 and 3 are shown in Figures 30, 31 and 32, respectively. Each area is 32 ft (9.7 m) long by 8.5 ft (2.6 m) wide and 1 ft (0.3 m) apart from the curb.

4.2.1. Visual Inspection

After removing the pavement/membrane system and sand blasting, the exposed concrete was visually inspected. On test section 1, the repaired areas were darker than the original concrete in color (see Figures 10 and 38). However, no defects could be visually detected in the area. On test section 2, the repaired areas as well as three spalls and two exposed rebars located in two of the spalled areas were detected as illustrated in Figure 39 (see also Figure 11). It is interesting to note that the defective areas were near the boundaries of the original and repaired concrete. On test section 3, no repaired concrete was located on the exposed concrete and no distress in the concrete could be visually detected.

4.2.2. Chain Dragging

The delaminations as detected by chain dragging the exposed concrete of test sections 1 and 2 are shown in Figures 38 and 39, respectively. The delaminated areas are almost always in the boundaries of the original and repaired concrete. The delaminations count for approximately 8% and 10% of the exposed area (32 ft or 9.7 m by 8.5 ft or 2.6 m) for test sections 1 and 2, respectively. As mentioned earlier (Section 4.1.2), the defects under the AC overlay on test section 2 and at the area designated for pavement removal as detected by chain dragging AC overlay (see Figure 33) was approximately 9%. Nevertheless, the patterns of the defects in Figures 33 and 39 are not completely the same. This may be due to possible errors in marking and mapping these areas. Chain dragging the exposed concrete on test section 3 did not show any delaminations in the concrete.

4.2.3. Electrical Potential Measurement

The results of electrical potential testing as recorded in the field on the exposed concrete are given in Figures 40, 41 and 42 for test sections 1, 2 and 3, respectively. The distribution of the half-cell readings are shown in Figures 43, 44 and 45 for test sections 1, 2 and 3, respectively. The distribution curves include the half-cell measurements in the core locations as well (see Section 4.1.3.). Also given in Figures 43, 44 and 45 are the distribution curves of the half-cell readings taken on the locations of the three test sections in 1975 (for more information see Part One of this study).

<u>Test Section 1</u>: In the 1983 survey, the highest and lowest half-cell potentials were -0.35 and -0.01 volts, respectively, whereas in the 1975 survey, the highest and lowest potentials were -0.52 and -0.24 volts, respectively (see Figure 43). About 86% of the readings in 1983 were smaller than -0.20 volts, which is the lower limit of the active corrosion of steel in concrete. All of the readings taken in 1975 were higher than -0.20 volts. No

reading in 1983 was higher than -0.35 volts, which is the higher limit of the active corrosion of steel in concrete, whereas 50% of the readings in 1975 were higher than -0.35 volts (see Figure 43). Generally, shifting of the half-cell distribution curve for 1983 to the left of that for 1975 suggests lower corrosion activity at the time of testing in 1983.

Test Section 2: The 1983 survey showed the highest and lowest half-cell potentials of -0.43 and -0.03 volts, respectively. In 1975 the highest and lowest potentials were -0.62 and -0.29 volts, respectively (see Figure 44). About 60% of the readings in 1983 were smaller than -0.20 volts (lower limit of corrosion). All of the readings in 1975 were higher than -0.20 volts. About 7% of the 1983 readings were higher than -0.35 (higher limit of corrosion). This number for the 1975 survey was about 82% (see Figure 44). Here again, shifting of the half-cell distribution curve for 1983 to the left of the curve representing the 1975 survey suggests lower corrosion activity at the time of testing in 1983.

Test Section 3: The half-cell survey on the exposed concrete was conducted on May 11, one day after exposing the concrete. As given in Figure 42, the readings taken along the curb (row c) were mainly positive. Positive values of potential are generally indicative of dry concrete [13]. Due to this discrepancy, half-cell readings are presented in two different ways: first, considering that the positive readings are invalid and analyzing only the negative readings; and second, studying the range of half-cell potentials as well as the absolute values of the potentials -- in other words, adding a negative value to the half-cell readings (negative and positive) taken on the exposed concrete until the highest positive reading becomes zero.

The distribution of 1983 half-cell readings on the test section with and without positive readings are found in Figure 45. Assuming the positive readings are invalid, the 1983 survey gave maximum and minimum potential values of -0.19 and -0.01 volts, respectively. Taking positive readings under consideration, these values will be -0.35 and 0.00 volts or a shifting of 0.16 volts (only for those half-cell potentials taken on the exposed concrete) which is equal to the highest positive reading. Also presented in Figure 45 are maximum and minimum half-cell potentials of the 1975 survey, which were -0.70 and -0.33 volts, respectively. Disregarding the positive readings, all of the recorded half-cell values in 1983 are smaller than -0.20 (lower limit of corrosion). Considering positive readings, about 75% of the readings are smaller than -0.20 volts. In either case, no half-cell potential is higher than -0.35 volts (higher limit of corrosion), whereas in the 1975 survey (Figure 45), all of the half-cell potentials were higher than -0.20 volts and only 6% of them were smaller than -0.35 volts (i.e., 94% higher than -0.35 volts). Comparison of the 1975 half-cell distribution curves with those of 1983 again suggests lower corrosion activity at the time of testing in 1983.

4.2.4. Chloride Content Measurement

Location of chloride samples taken on the exposed areas are shown in Figures 38, 39 and 42 for test sections 1, 2 and 3, respectively. On test sections 1 and 2 the samples were taken from the repaired areas protected by the waterproofing system immediately after the repair job in 1976. On test section 3, sampling was done on the original concrete. Each test section had two chloride data points and each chloride data point gave the chloride content at two different depths at test sections 1 and 2, and at three different depths at test section 3. The results of the laboratory chloride analysis of the samples are given in Table 5 and illustrated in Figure 46. As indicated in Figure 46, for those samples taken from the patches, the chloride content increases with depth in the concrete, which is rather confusing. One explanation may be that while sampling at the depth of 1.0 in (2.5 cm) to 1.5 in (3.8 cm) the original concrete (contaminated) was hit, thus giving higher chloride content values for that depth. Assuming the original chloride content of the patch concrete to be about 0.30 lb/cy (0.18 kg/m^3) (as determined for the Roza Canal Bridge deck in Section 4.4.5), the maximum chloride increase at test section 1 at the depth of 1.0 in (2.5 cm) since 1976 will be (0.98) - (0.30) = 0.68 lb/cy (0.40 kg/m^3). Using the same procedure for test section 2, the maximum increase at the same depth will be (0.73) - (0.30) = 0.43 lb/cy (0.27 kg/m^3). To obtain the amount of chloride increase at test section 3, the results of the resampling in Table 4 could be used. Sample no. 1 in the table was located in test section 3 and only 11 ft (3.3 m) apart from the exposed concrete of the test section. As given in Table 4, the amount of chloride increase for sample no. 1 since 1975 is (5.20) - (3.45) = 1.75 lb/cy (1.03 kg/m^3) at the depth of 1.5 in (3.8 cm). The latter result is somewhat contradictory to what is illustrated for test section 3 in Figure 4. For example, the chloride profile of sample 6,7-c at test section 3, which is about 15 ft apart from sample 1 in Table 4, shows a maximum chloride content of only 0.83 lb/cy (0.50 kg/m) (compared with 5.20 lb/cy of sample 1). The other sample in test section 3 (sample 2-d) also shows a maximum chloride content of 0.86 lb/cy (0.51 kg/m^3). In addition, the trend of chloride profile for sample 6,7-c is quite different from that of sample 2-d (see Figure 46). In other words, there is no uniform behavior for the profile curves.

4.2.5. Pachometer Survey

Results of the pachometer survey on the exposed concrete and location of the measurements are presented in Figures 40, 41 and 42 for test sections 1, 2 and 3, respectively. All pachometer readings are converted into inches considering a correction of 0.2 in (0.5 cm) for a pachometer depth of 2 in (5.1 cm). The correction was found when sampling concrete for chloride analysis on the exposed concrete at test section 3 (chloride sample 6,7-c) and reaching the rebar. According to the pachometer survey, the average cover thickness on test section 1 is about 1.5 in (3.8 cm) with a maximum and minimum of 1.8 and 1.4 in (4.6 and 3.6 cm), respectively. On test section 2 the average cover thickness is about 1.2 in (3.0 cm) with a maximum and minimum of 1.4 and 1.0 in (3.6 and 2.5 cm), respectively. For test section 3 the pachometer showed an average cover thickness of about 2.4 in (6.1 cm) with the highest and lowest thicknesses of 2.5 in (5.6 cm) and 2.2 in (6.4 cm), respectively.

4.3. Columbia River Bridge (285/10), Testing Newly Waterproofed Deck

On May 11, 1983, the exposed concrete on test sections 1, 2 and 3 were waterproofed and paved with a membrane and asphalt concrete (see Section 3.9). The following is a description of the tests conducted during this period.

4.3.1. Electrical Resistivity Testing of Newly Installed Membrane

After installing the system (A) waterproofing membrane on the exposed concrete (see Section 3.9), it was immediately tested for resistivity (see

Figure 29). The results for test sections 1, 2 and 3 are found in Figures 47, 48 and 49, respectively. Except at one location on test section 1 (see Figure 47), which showed a resistivity of 19,000 ohms, the readings were either infinity or very high values, which were an indication of satisfactory waterproofing membrane at the time of application.

4.3.2. Electrical Resistivity Testing of Newly Installed Pavement/ Membrane System

The electrical resistivity readings taken on the newly paved asphalt concrete of test section 1 were generally infinity, except for two locations close to the old pavement which showed 180,000 and 150,000 ohms. It was possible that the relatively low readings in this case were due to excessive moisture on the pavement caused by the steel-drum compactor while compacting the mixture. This moisture might have shorted the circuit in those areas.

4.3.3. Chain Dragging Newly Paved Asphalt Concrete

Chain dragging newly paved asphalt concrete resulted in a low amplitude sound to be heard all over the area, thus making the detection of defects impossible. This was probably because the asphalt concrete was not set up enough to allow sound to be rebound.

4.4. Roza Canal Bridge (82/106) Testing Asphalt Covered Deck

The bridge was built in 1969 and the concrete deck was waterproofed at the same time. The waterproofing system from top to bottom consists of a layer of class B asphalt concrete, a non-woven polypropylene fabric (Petromat), and asphalt emulsion applied on the concrete. Part One of this study gives more background information regarding this bridge.

Testing was conducted on April 19. Weather conditions were clear with air temperatures in the low 70s°F (21°C). A test section 105 ft (31.8 m) long by 18 ft (5.5 m) wide was located on almost the entire outside lane and shoulder of the 82/106 L/S bridge. The test section and its test reading setup are shown in Figure 50.

4.4.1. Visual Inspection

Asphalt concrete on the test section was almost in good condition and no defects could be detected. The pavement in the traffic lane looked highly dense and rich in binder. Concrete of the curbing was in good shape with no apparent deterioration. Examining the deck from the underside was difficult. However, a portion of the deck (southwest corner) was visually inspected from the underside and efflorescence could rarely be detected (see Figure 9-2). Generally, no major key pointers to defective concrete under the asphalt overlay could be found by visual inspection.

4.4.2. Chain Dragging

Chain dragging the asphalt overlay on the test section did not detect any defects under the pavement.

4.4.3. Electrical Resistivity Testing

The results of the electrical resistivity testing recorded in the field are shown in Figure 51. The distribution of the resistivity readings is given in Figure 52. As indicated in the figure, about 90% of the resistivity readings at the test section are smaller than 250,000 ohms (WSDOT passing level for newly installed pavement/membrane systems is 30% lower than 250,000 ohms). As shown in Figure 52, about 75% of the readings are also smaller than 100,000 ohms (lower limit for a satisfactory waterproofing membrane).

4.4.4. Coring Program

Location of the core samples of AC and membrane are shown in Figure 50, and the data from the coring program are tabulated in Table 6.

Membrane: The average AC and membrane thickness of the core samples actually measured at the site was about 0.9 in (2.3 cm). The bond of membrane to AC overlay was recorded good 33% of the time, fair 40%, and poor 27%. The bond between membrane and concrete was recorded good 13% of the time, fair 47% and poor 33%. Generally, those core samples in the traffic lane (rows a and b) showed good to fair bonds in the interface, whereas the interface bond for those in the shoulder areas (rows c and d) was fair to poor. Examination of the cored membranes at the WSDOT Materials Laboratory indicated that 40%, 27% and 7% of the membrane samples had severe, moderate, and slight pinholes, respectively (a total of 74% of the samples). Among the membrane samples taken at the traffic lane (rows a and b), 86% showed pinholes, whereas at the shoulder (rows c and d), 63% of the samples had pinholes.

Concrete Deck: The concrete surface in the core locations immediately after removing the membrane was severely wet 7% of the time, moderately wet 27%, and slightly wet 20% (a total of 54% of the core locations). All of the cores located along the curb (row d) showed moisture on the concrete surface. However, at the traffic lane and at the shoulder but close to the traffic lane (rows a, b and c), only 30% of the cores had moisture on the concrete surface after removing the membrane. The exposed concrete in the core locations did not show any surface distress. No delaminations in the concrete could be detected by tapping the exposed concrete in the core locations with a hammer. The average concrete cover thickness as measured by the pachometer over the AC overlay (considering an average correction of 0.4 in or 1.0 cm for any depth measured by the pachometer as determined in the field by actual coring and measuring at the location of core 2-d and ground) was about 1.8 in (4.6 cm).

Electrical Potential Testing: The results of electrical potential testing as recorded in the field are given in Table 6. The distribution of the half-cell potential values measured in the core locations is illustrated in Figure 53. The highest and lowest negative half-cell potentials were -0.103 and -0.040 volts, respectively. As indicated in Figure 53, the readings were considerably lower than -0.20 volts (lower limit of corrosion).

4.4.5. Chloride Content Measurement

Location of chloride data points on the test section, including both chloride content at the level of rebar and chloride content versus concrete depth, is given in Figure 50. Laboratory results of chloride analysis are given in Tables 6 and 7 for chloride content at the level of rebar and chloride content versus concrete depth, respectively. The chloride content depth profile is plotted in Figure 54. As shown in the figure, chloride has penetrated into the concrete deck in the area close to the curb (sample 2-d). However, chloride penetration in the area close to the centerline (sample 11-a) is much lower. A possible reason for this may be a higher density of the AC overlay in the traffic lane in conjunction with the cross-fall of the deck, which leads the water to the curb side. Figure 54 also suggests that the original chloride content in the concrete should be in the range of 0.30 to 0.45 lb/cy (0.18 to 0.27 kg/m^3). Taking this under consideration, the penetration of chloride into the concrete at the area close to the curb (sample 2-d) and at a depth of 0.5 to 1 in (1.3 to 2.5 cm) is about (1.13) - (0.30) = 0.83 lb/cy (0.49 kg/m^3). Table 6 indicates that the highest chloride content measured at the level of rebar is 0.69 lb/cy (0.41 kg/m^3) (samples 2-d), which is about (0.69) - (0.30) = 0.39 lb/cy (0.17 kg/m^3) higher than the original chloride content of the concrete. The rest of the samples in Table 6 (samples 6-b, 7-d, 10-c, 14-c, and 18-a) almost have a chloride content in the range near that obtained for the original chloride content, except sample 11-a, which has a chloride content of about 0.24 lb/cy (0.14 kg/m^3) above that of the original chloride content.

The total chloride content corrosion threshold for a concrete deck is 0.20% based on the cement weight (assuming that 75% of chloride is soluble, see [14]). Relating this 0.20% Cl to the concrete of the deck, with a cement content of 660 lb/cy (389 kg/m^3) (class AX mix), the total chloride content corrosion threshold is approximately (660)x(0.20%) = 1.32 lb/cy (0.78 kg/m^3). The results of chloride measurement in Table 6 indicate that the concentration of chloride at the level of rebar and in the tested areas is lower than the threshold value.

Because of the small values of chloride content at the level of rebar associated with the small values of the electrical potential of the rebar (see Table 6), a meaningful correlation between chloride content and electrical potential does not appear to be possible in this case.

4.5. Ebey Slough Bridge (5/650 W), Testing Asphalt Covered Deck

The Ebey Slough Bridge was built in 1954. In 1973 the deck was waterproofed with an asphalt/membrane protective system. The system from top to bottom consisted of a layer of class B asphalt concrete, a non-woven polypropylene fabric (Petromat), and asphalt emulsion applied on the concrete surface. No major repair was done on the deck prior to the waterproofing (for more background information, see Part One of this study).

Testing was conducted on April 5, 6 and 7. Weather conditions were clear, overcast, and overcast for April 5, 6 and 7, respectively, and it had rained the night before April 7. The air temperature was at 60 F (16 C) for the period of testing. A test section 150 ft (45.5 m) long by 13 ft (3.9 m) wide, as shown in Figure 55, was located on the inside lane and shoulder of the southbound lane for a complete set of tests. The test section began with expansion dam no. 3 and ended with expansion dam no. 1 (see Figure 55). This configuration reduced the number of rebar exposures (ground) for electrical resistivity and potential testing. Figure 56 shows the test reading setup on the test section. In addition, the testing program included extra core sampling on the inside lane southbound and south of the test section (see

Figures 55 and 57). This area was located on grade and the AC overlay was heavily patched, possibly due to concrete deterioration (see Figure 6).

4.5.1. Visual Inspection

The asphalt overlay on the test section was raveling in the wheel paths up to 0.5 in (1.3 cm). A few small areas 2-3 in (5.1 - 7.6 cm) deep could be detected in the right wheel path. Transverse cracks on the AC overlay along the expansion dams could be seen (see Figure 7). The concrete of the sidewalk and railing was in fair condition. The curb was scraped on the edge, possibly due to traffic abrasion. A portion of the deck was inspected from the underside and considerable transverse efflorescence could be noticed.

4.5.2. Chain Dragging

No defects under the AC overlay at the test section could be detected by chain dragging the AC overlay, except in the areas along the expansion dams.

4.5.3. Electrical Resistivity Testing

In Figure 58 are shown the results of electrical resistivity testing as recorded in the field. The distribution of the resistivity values is given in Figure 52. As presented in Figure 52, about 85% of the resistivity values at the test section were smaller than 250,000 ohms (WSDOT passing level for newly installed pavement/membrane system is 30% for 250,000 ohms). It can also be seen in Figure 52 that about 83% of the resistivity readings are lower than 100,000 ohms, which is the lower limit for a satisfactory membrane. The distribution curve also indicates that the majority of the readings (about 75%) were smaller than 10,000 ohms, which is considerably low.

4.5.4. Coring Program

The location of the core samples of AC and membrane at the test section, as well as those near the AC patches on grade, are given in Figures 55 and 57, respectively. The tabulated data from the coring program are shown in Table 8.

<u>Membrane</u>: The average AC overlay and membrane thickness actually measured in the field was about 1.5 in (3.8 cm) and 1.1 in (2.8 cm) for the test section and the patched area, respectively. At the test section the bond between membrane and AC overlay was recorded as good 24% of the time, fair 59% of the time, and poor 18% of the time. The bond of membrane to the concrete deck at the test section was good 18% of the time, fair 41%, and poor 41%. In general, with a few exceptions, those core samples in the traffic lane (rows a and b) showed good to fair bonding in the interface, whereas the interface bond for those in the shoulder area (row c) was fair to poor. On grade and near the patched areas, the bond in the interface was recorded as poor for all core samples. Laboratory examinations of the cored membrane samples indicated that 26%, 26% and 21% of the membrane samples in the core locations had severe, moderate and slight pinholes, respectively (a total of 73% of the samples). Among the membrane samples taken at the traffic lane (rows a and b), 64% showed pinholes, whereas all of the membrane samples taken at the shoulder (row c) had pinholes. This result is contradictory to that of the Roza Canal test site, at which the traffic lane had relatively more pinholes than the shoulder area. One possible reason might be that the core locations at the

traffic lane of the Ebey Slough test site did not coincide with the wheel paths.

Concrete Deck: The exposed concrete surface in the core locations was severely wet 14% of the time, moderately wet 19%, and slightly wet 43% (a total of 76% of the core locations). Among the core locations along the curb (row c), 83% showed moisture on the concrete surface. In the traffic lane (rows a and b), this number was 73%. On the test section, 12% of the core locations (2 out of 17) showed surface distress in the concrete. On the patched area, 25% of the core locations (1 out of 4) showed delaminations as indicated by a hammer. The average cover thickness as measured by the pachometer over the AC overlay (considering a correction of 0.7 in (1.8 cm) for a total pachometer depth of 2.8 in (7.1 cm) as determined by comparison of sample 20-c and ground 3) was 2.0 in (5.1 cm) and 1.3 in (3.3 cm) for the test section and the patched area, respectively.

Electrical Potential Testing: The results of electrical potential testing as recorded in the field are given in Table 8. The distribution of half-cell potentials measured in the core locations is illustrated in Figure 53. The highest and lowest negative readings on the test section were -0.114 and -0.011 volts, respectively. As indicated in Figure 53, the half-cell potentials on the test section are significantly lower than the lower limit for active corrosion of steel in concrete (-0.20 volts). The distribution of the potentials on the Ebey Slough test section is more or less like that of the Roza Canal test site. Nevertheless, on the Ebey Slough Bridge deck, where the AC patches are located, the half-cell potentials were significantly higher than those on the test section. The potential distribution curve for this area is given in Figure 53, in which the curve is located to the right of the one representing the Ebey Slough test section. The highest half-cell potential at the patched area (-0.370 volts) was taken directly over one of the asphalt patches, knowing that there was no membrane (dielectric material) installed underneath. The rest of the half-cell potentials at the patched area were taken in the core locations around the patches (see Figure 56 for core locations).

4.5.5. Chloride Content Measurement

Location of chloride content data points at the level of rebar at the test section, as well as the patched area, is given in Figures 56 and 57, respectively. Location of the resamples of the 1974 chloride measurement on the deck is given in Figure 55. Results of the laboratory chloride analysis are presented in Tables 8 and 9 for chloride at the level of rebar and resamples of the 1974 survey, respectively. As given in Table 9, chloride has penetrated into the concrete deck since 1974 with an average amount of 0.59 lb/cy (0.35 kg/m^3) or 33% (average of 4 measurements at different locations at a depth of 0.5 in or 1.3 cm to 1.25 in or 3.2 cm in the concrete).

No comparison can be made for those chloride contents obtained at the level of rebar (Table 8) since no baseline data are available. However, if the total chloride content corrosion threshold of 0.2% based on the weight of cement is used (see [14]), the threshold for the concrete of the deck having a cement content of 610 lb/cy (360 kg/m^3) (class A mix) will be: (610)x(0.20) = 1.22 lb/cy (0.72 kg/m^3). The results in Table 8 indicate that the concentration of chloride at the level of rebar exceeds the threshold value.

A correlation may be possible between chloride content at the level of rebar and the corresponding half-cell potentials for the Ebey Slough test sight, as shown in Figure 59.

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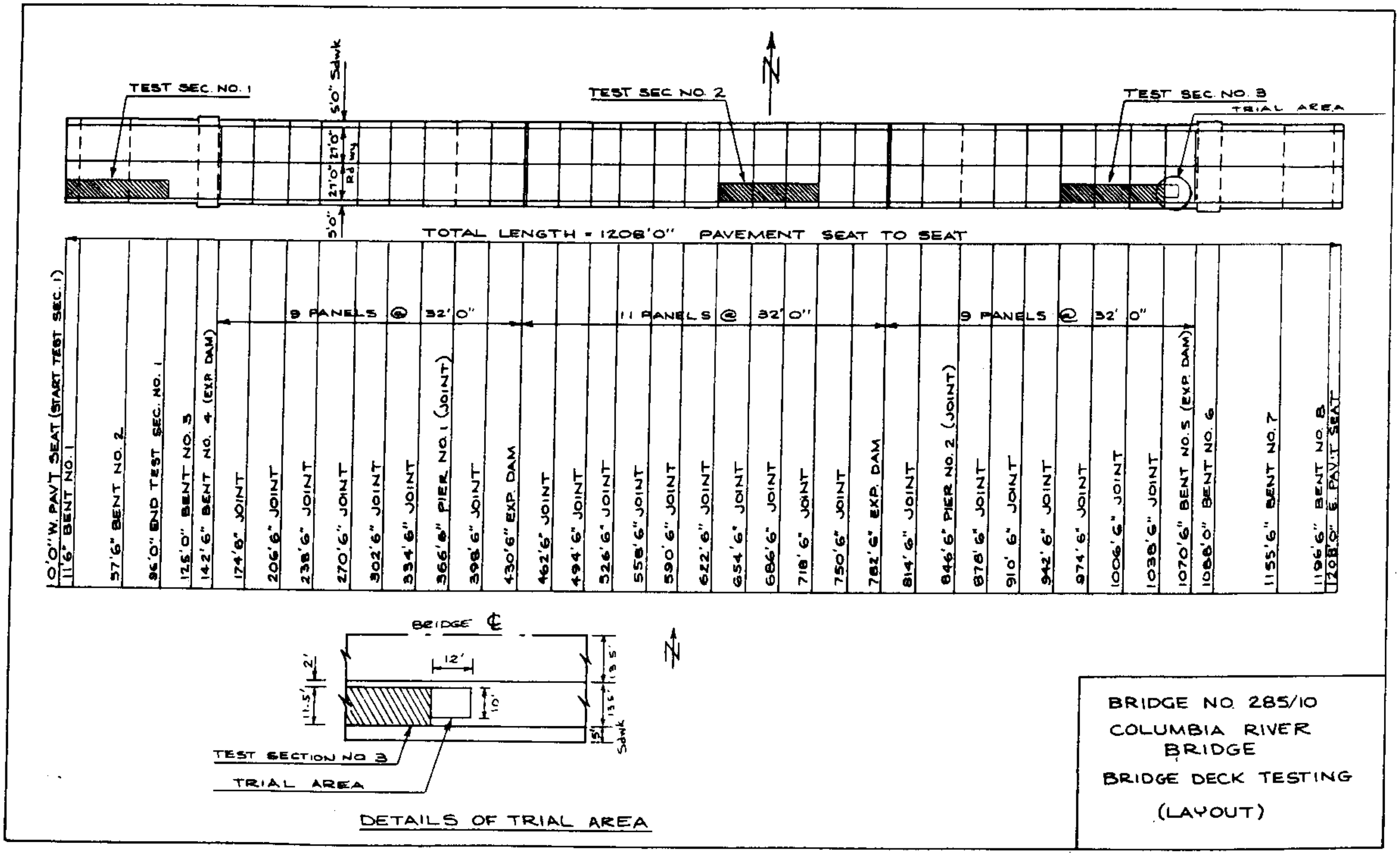

Figure 1. Layout of Initial Test Sections on Bridge 285/10 and Location of Trial Pavement Removal

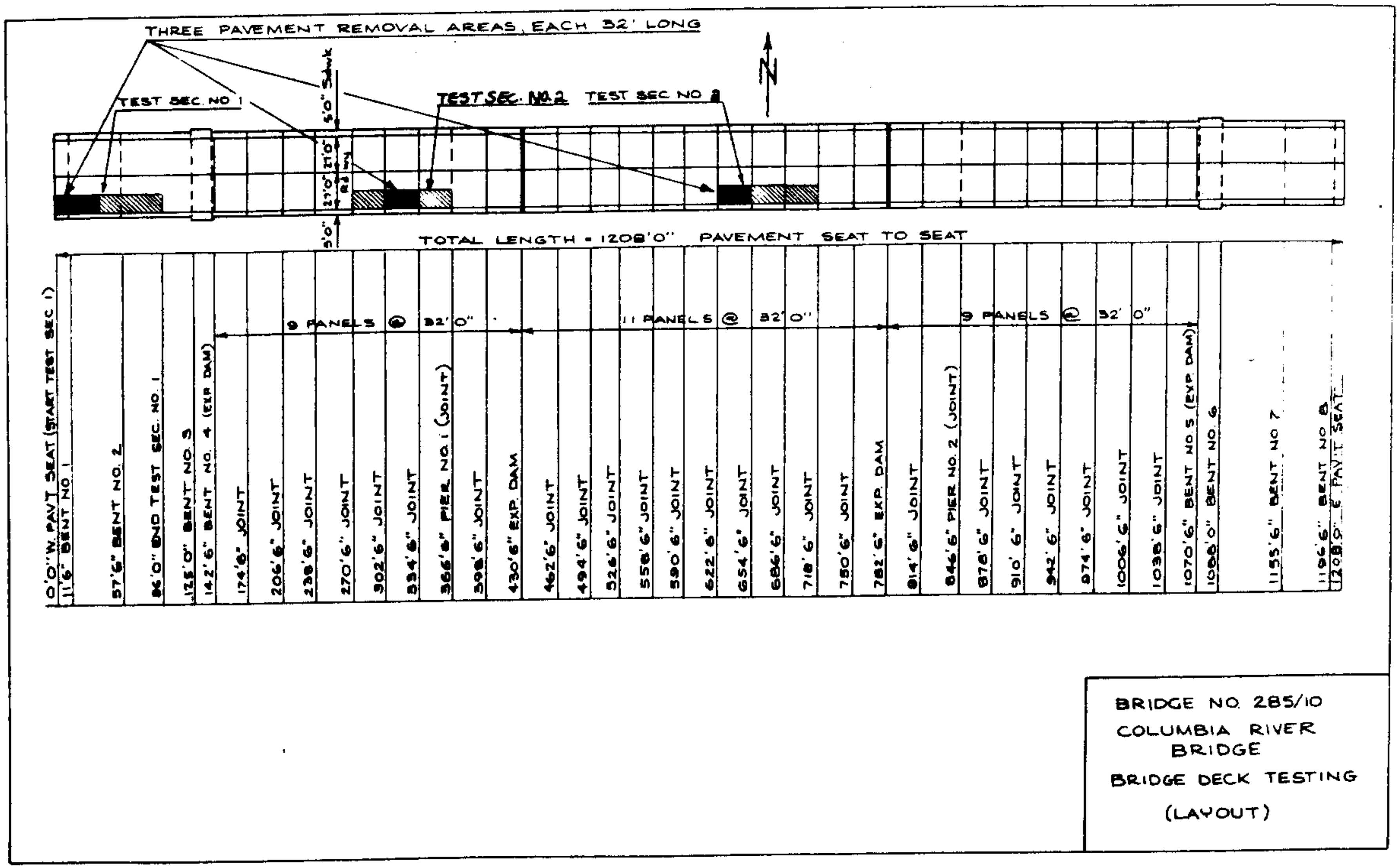

Figure 2. Layout of Revised Test Sections on Bridae 285/10 and Locations of Concrete Deck to be Exposed

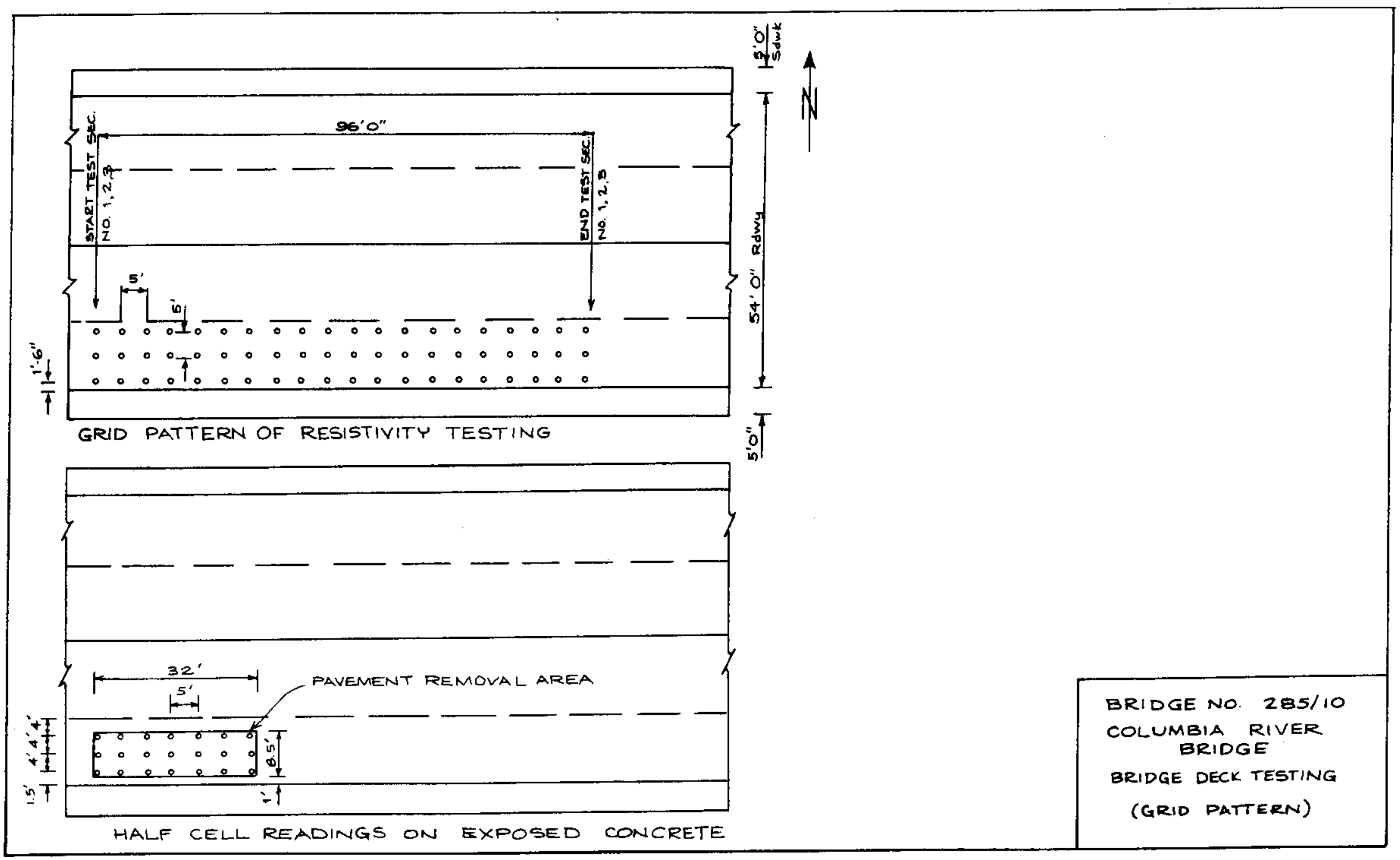

Figure 3. Grid Pattern of Resistivity Testing on AC and Half-Cell Testing on Exposed Concrete in the Revised Testing Plan

			None	Slight	Moderate	Severe	Good	Fair	Poor	Remarks
Interface	Membrane to Asphalt	Bond								
		Moisture								
	Membrane to Concrete	Bond								
		Moisture								
Membrane Appearance		Cracking								
		Brittleness								
		Thickness Uniformity								
		Pinholes								

Figure 4. Data Form for AC and Membrane Core Samples

LOCATION	ACTIVITY
Entire Test Section on AC	1. Visual survey 2. Chain dragging 3. Resistivity reading
Portion of Test Site Not to be Exposed	1. Core sampling AC and membrane (dry) 2. Half-cell reading in core locations
Exposed PCC Only	1. Visual survey 2. Chain dragging 3. Pachometer survey 4. Half-cell reading 5. Chloride content measurement* 6. Membrane installation 7. Resistivity testing on new membrane 8. Paving 9. Chain dragging new AC 10. Resistivity testing on new AC

*not limited to exposed concrete

Figure 5. Summary of Testing Activities on Each Test Section

Figure 6. AC Patches on Ebey Slough Bridge Deck on Grade

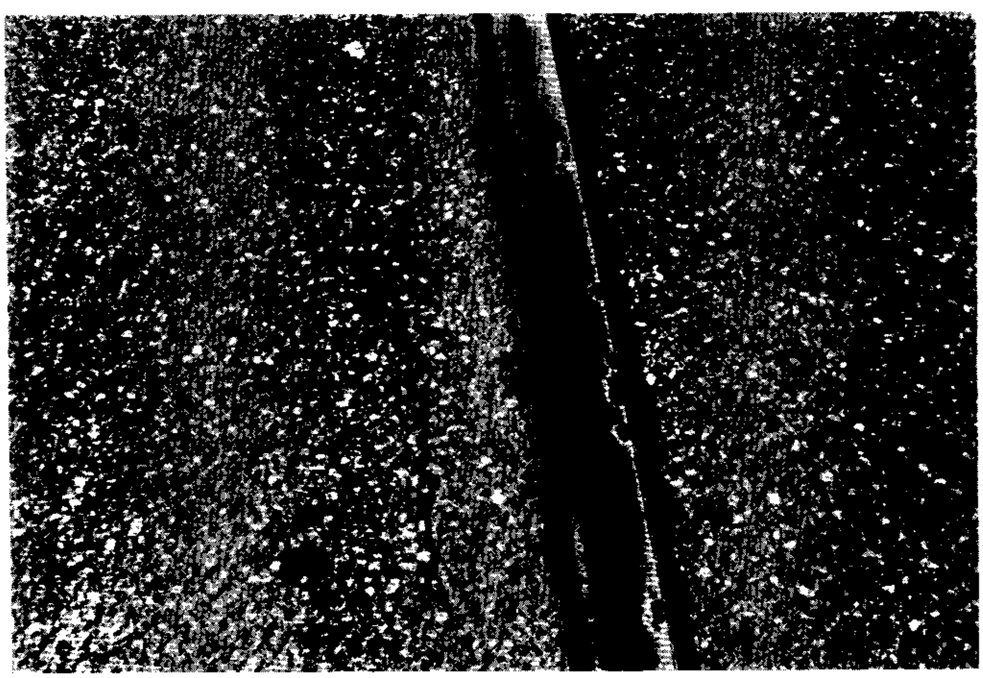

Figure 7. Transverse Crack Along Expansion Dam on Ebey Slough Bridge Deck

Figure 8. Transverse Crack Above Slab Joint Covered with Asphalt Overlay (Columbia River Bridge Deck)

Figure 9. Curb Scraped on Edge (Columbia River Bridge Deck)

Figure 9-1. Transverse Efflorescence on Underside of Ebey Slough Bridge Deck

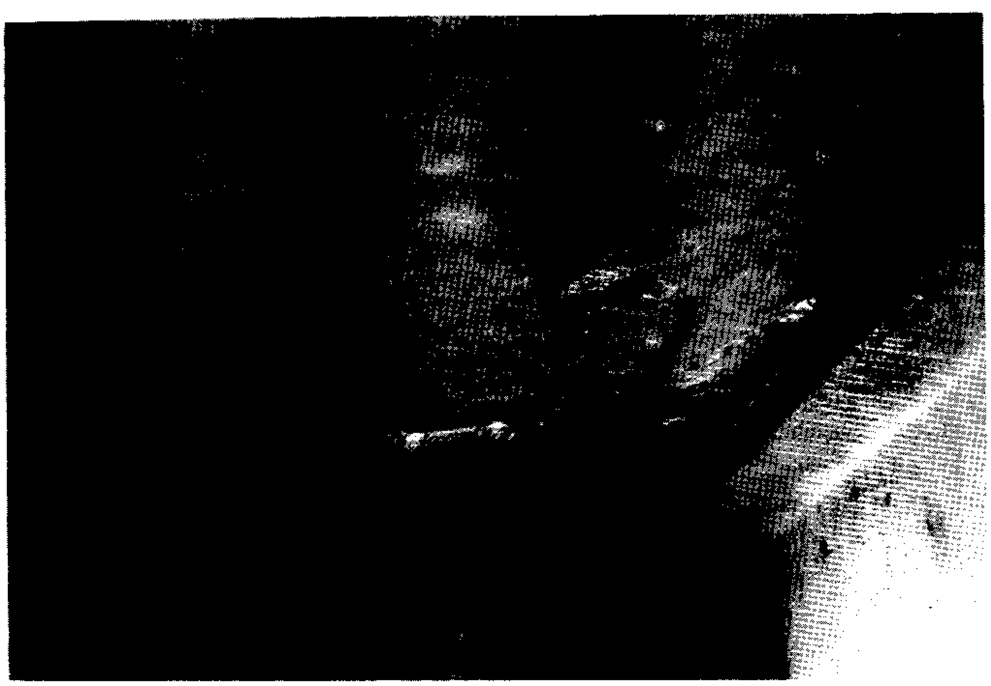

Figure 9-2. Efflorescence on Underside of Roza Canal Bridge Deck (see conjunction of beam and diaphragm)

Figure 10. Exposed Concrete After Sand Blasting (note previous repaired areas in darker color) (Columbia River Bridge Deck, test section 1)

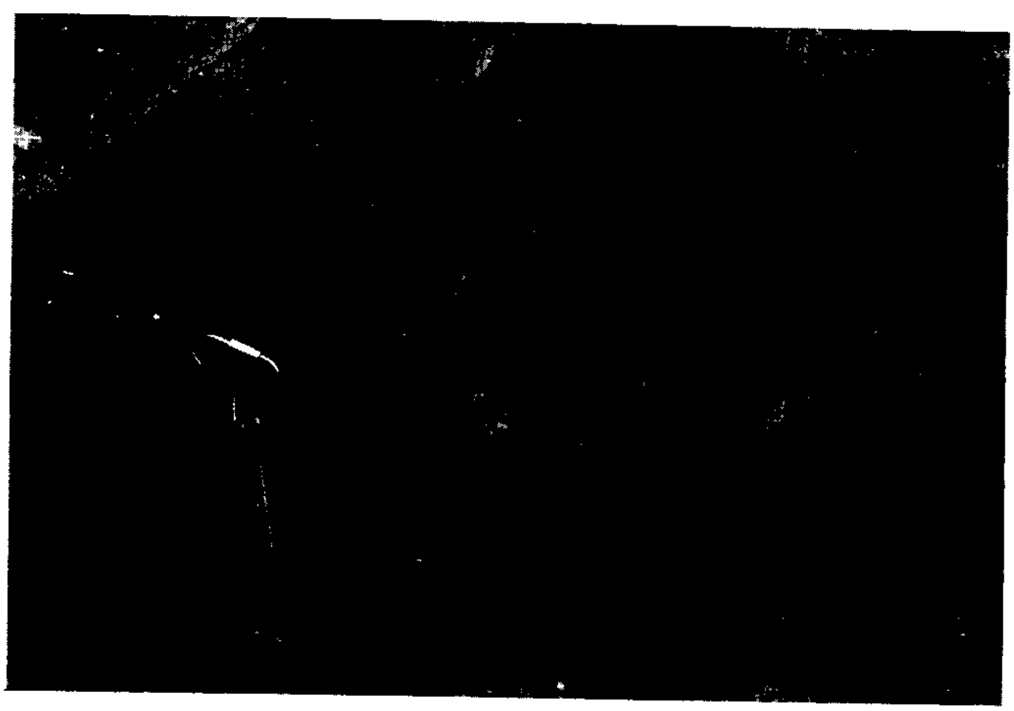

Figure 11. Spalling Area on Exposed Concrete (Columbia River Bridge Deck, test section 2)

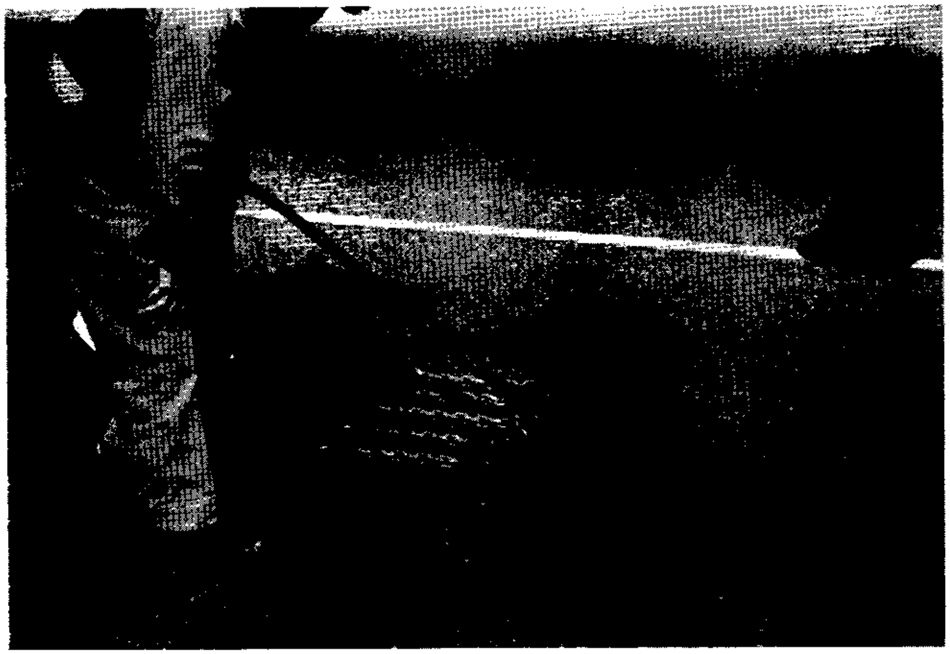

Figure 12. Chain Dragging AC Overlay (Columbia River Bridge Deck)

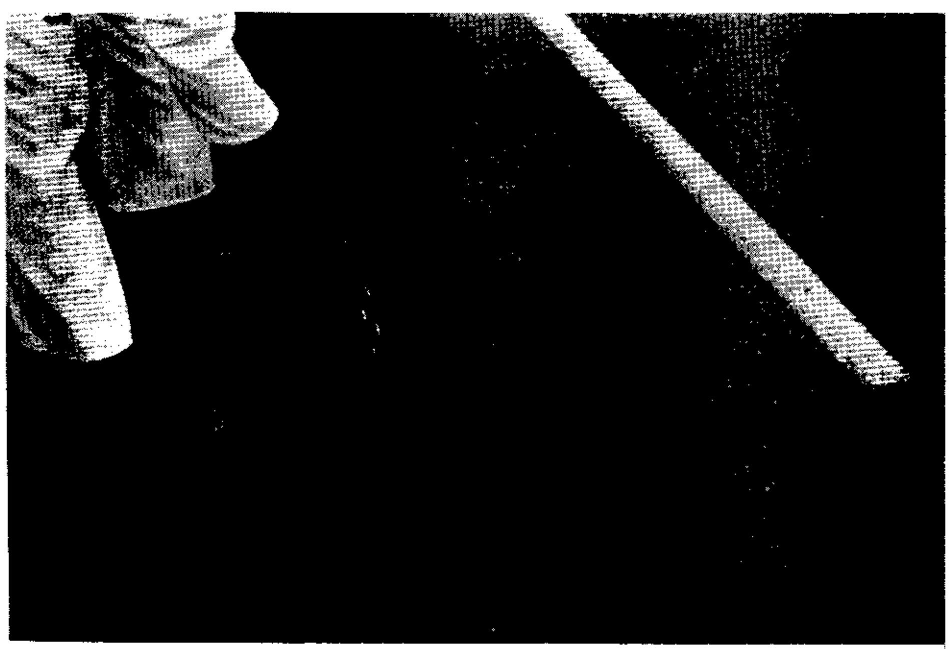

Figure 13. Defining Boundaries of Defective Areas Under AC Overlay (Columbia River Bridge Deck)

Figure 14. Sampling Concrete for Chloride Analysis in a Core Location Using a Rotary Hammer (Roza Canal Test Site)

Figure 14-1. Obtaining Powdered Concrete in a Core Location for Chloride Analysis (Roza Canal Test Site)

Figure 14-2. Sampling Concrete for Chloride Analysis on Exposed Concrete with Rotary Hammer (Columbia River Bridge Deck, test section 2)

Figure 15. Taking Electrical Resistivity Reading on AC Overlay (Ebey Slough Bridge Test Site)

Figure 16. Wetting Grid Points with Water and Wetting Agent Solution Prior to Electrical Resistivity Testing (Ebey Slough Bridge Test Site)

Figure 17. Direct Electrical Continuity to Top Mat Reinforcing Steel by Means of Compression Type Ground Clamp (Columbia River Bridge Deck)

Figure 18. Core Sampling AC Overlay and Membrane by Truck-Mounted, Nitrogen-Cooled Coring Rig (Columbia River Bridge Deck)

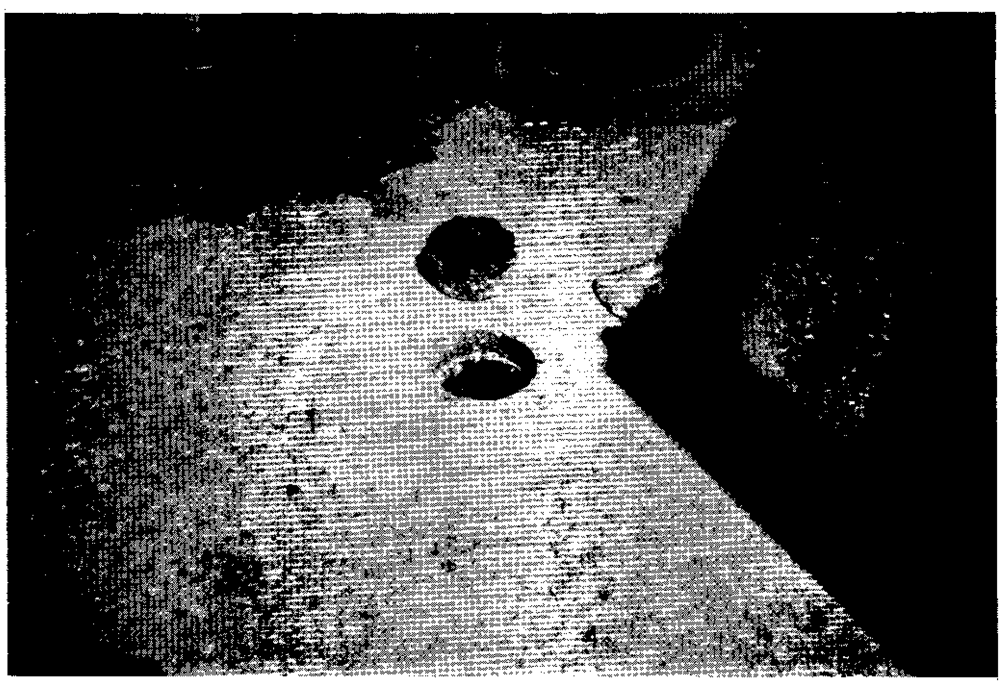

Figure 19. AC Overlay Core Sample Stuck to Membrane (Rubberized Asphalt) Removed from Core Location by Hammer and Chisel (Columbia River Bridge Deck)

Figure 19-1. Waterproofing Core Location with a Piece of Membrane (Roza Canal Test Site)

Figure 19-2. Patching Core Location with a Cold Asphalt Mix (Roza Canal Test Site)

Figure 20. Half-Cell Measurement on Exposed Concrete in a Core Location (Ebey Slough Bridge Deck)

Figure 20-1. Half-Cell Measurement on Exposed Concrete Deck (Columbia River Bridge Deck, test section 3)

Figure 21. Locating a Rebar by Pachometer to Provide Ground (Columbia River Bridge Deck)

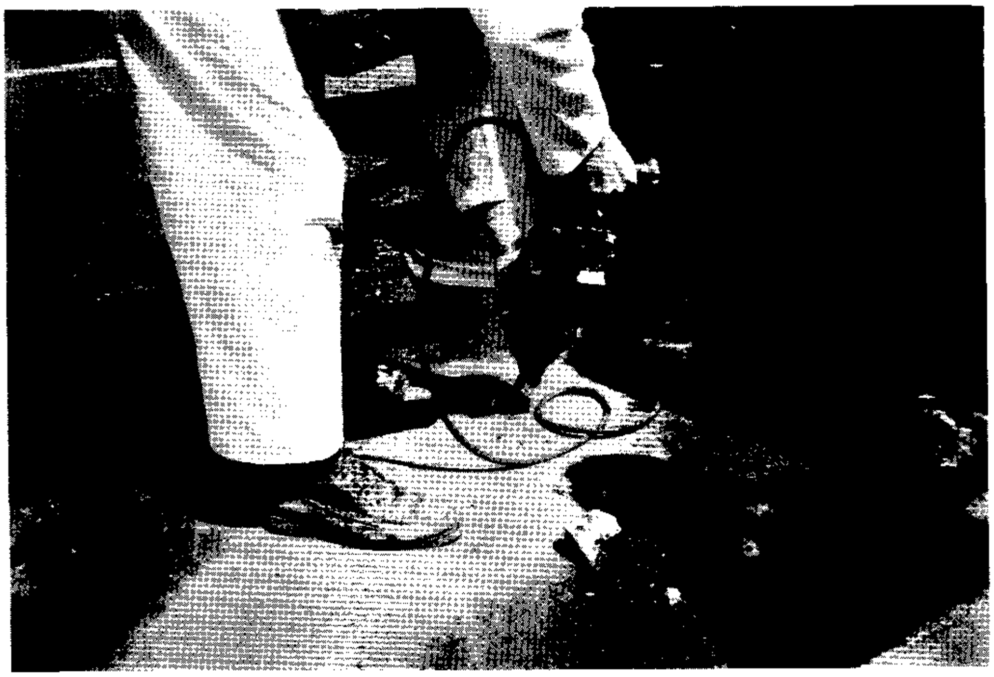

Figure 22. Digging Concrete in Core Location to Reach Rebar for Ground (Columbia River Bridge Deck)

Figure 23. Removing Pavement/Membrane System Using a Jackhammer (Columbia River Bridge Deck)

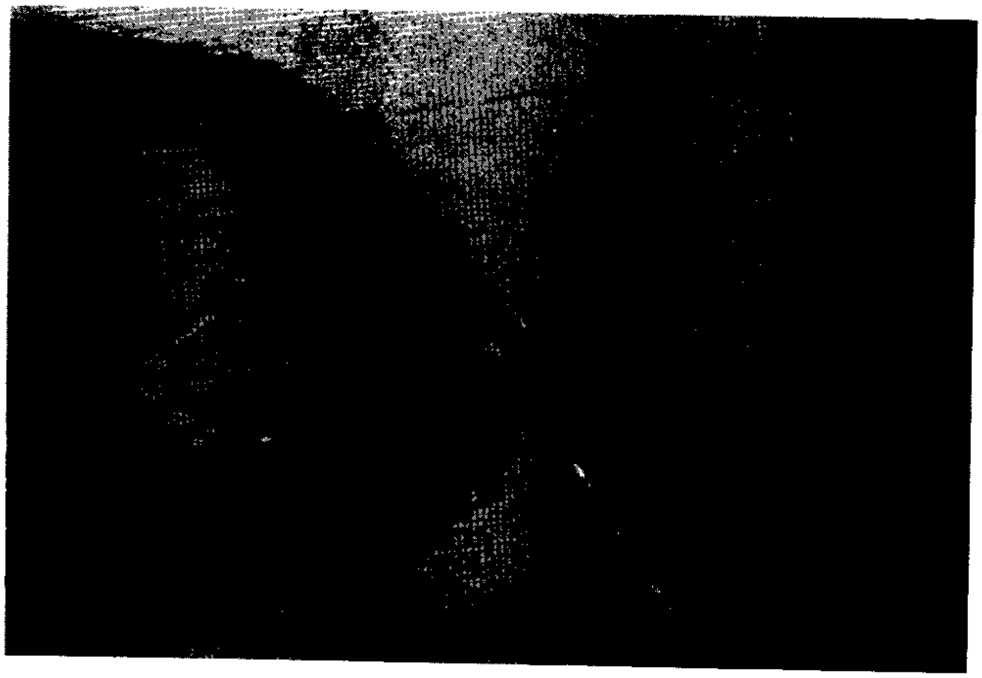

Figure 24. Sand Blasting Exposed Concrete Surface (Columbia River Bridge Deck, test section 1)

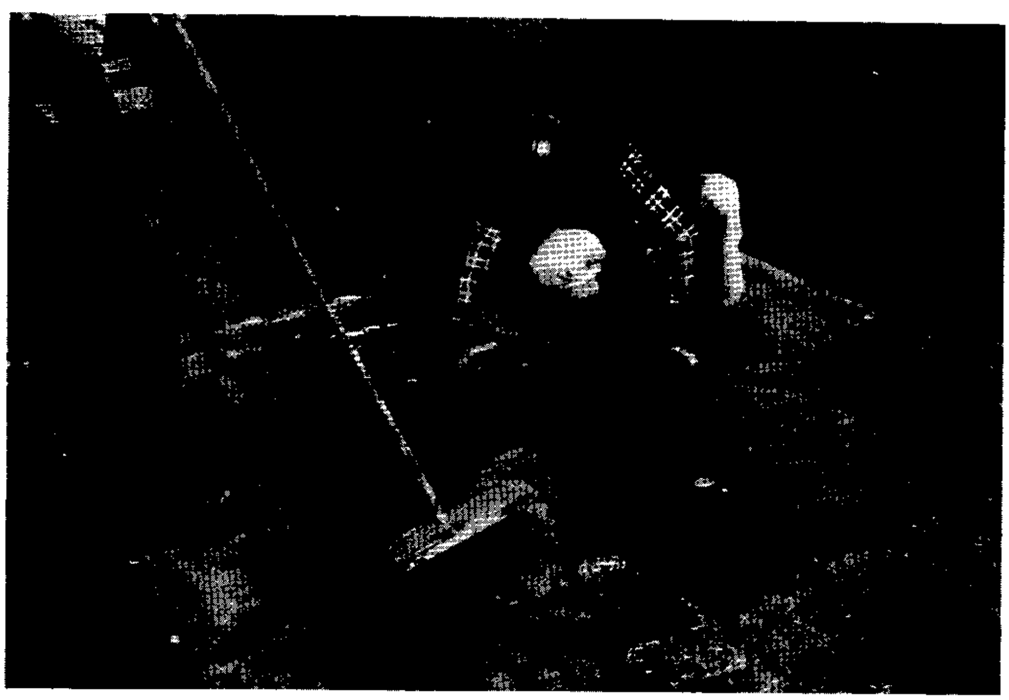

Figure 25. Patching a Spalled Area on Exposed Surface Prior to Waterproofing with Membrane (Columbia River Bridge Deck, test section 2)

Figure 25-1. Sealing Perimeter of Exposed Area with Elastomeric Mastic After Application of Glue (Primer) on Concrete Surface (Columbia River Bridge Deck, test section 2)

Figure 26. Unrolling Membrane on Exposed Concrete Surface (Columbia River Bridge Deck)

Figure 27. Removing Wrinkles and Air Bubbles with a Shovel While Unrolling Membrane on Concrete Surface (Columbia River Bridge Deck)

Figure 28. Using a Hand Roller to Bring Membrane Completely in Contact with Concrete Surface (Columbia River Bridge Deck)

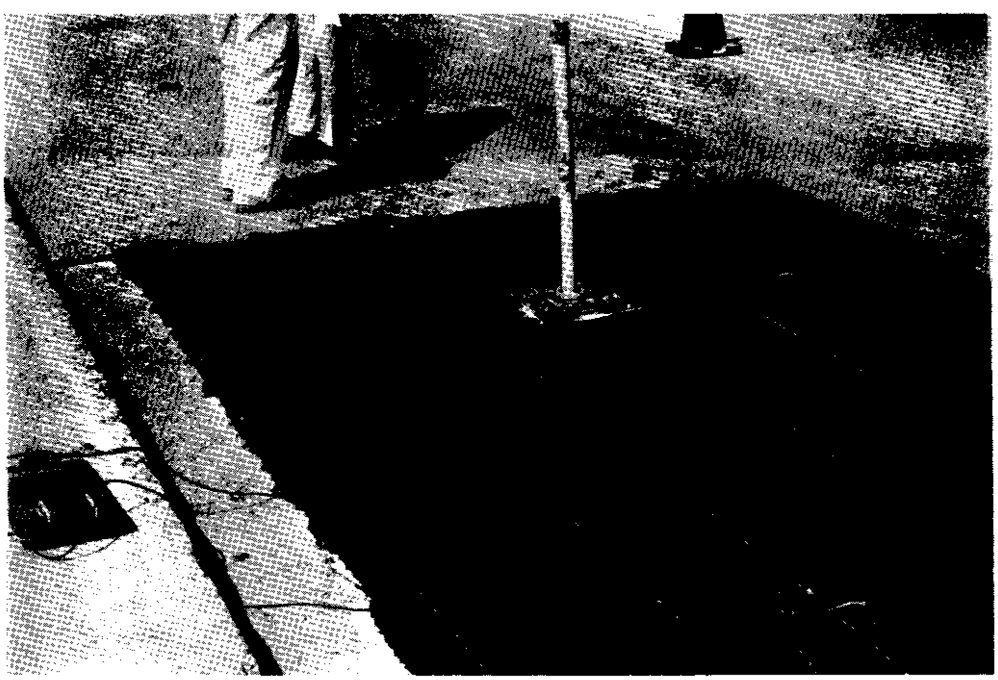

Figure 29. Conducting Electrical Resistivity Testing on Newly Installed Membrane (Columbia River Bridge Deck)

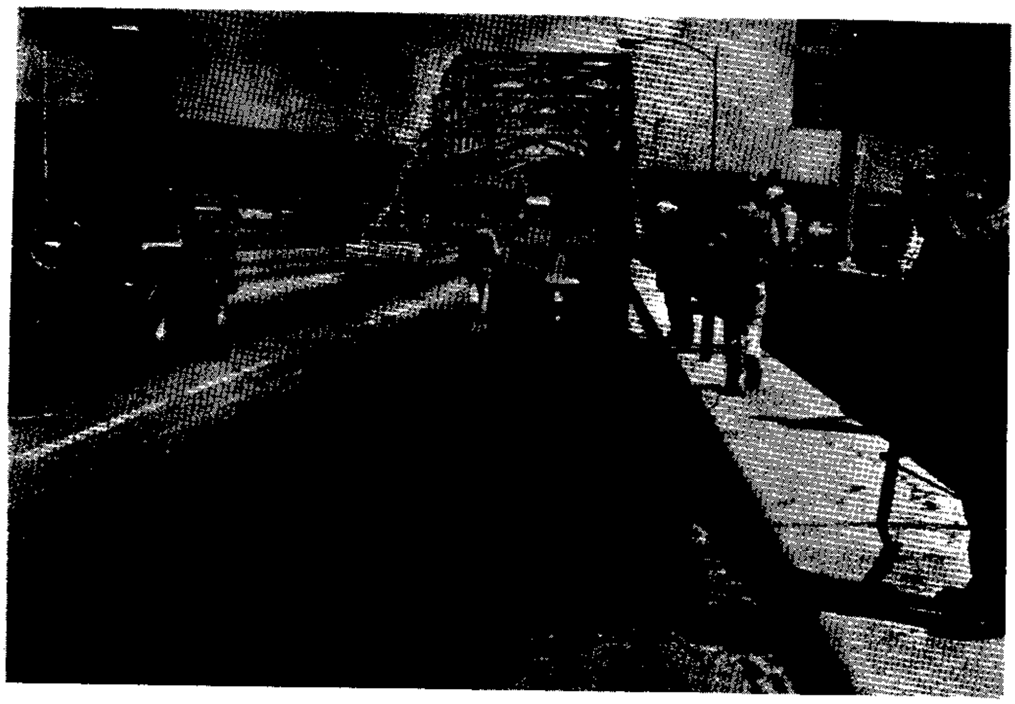

Figure 29-1. Compacting Asphalt Concrete Mix Applied on Newly Installed Membrane by Steel-Drum Compactor (Columbia River Bridge Deck, test section 1)

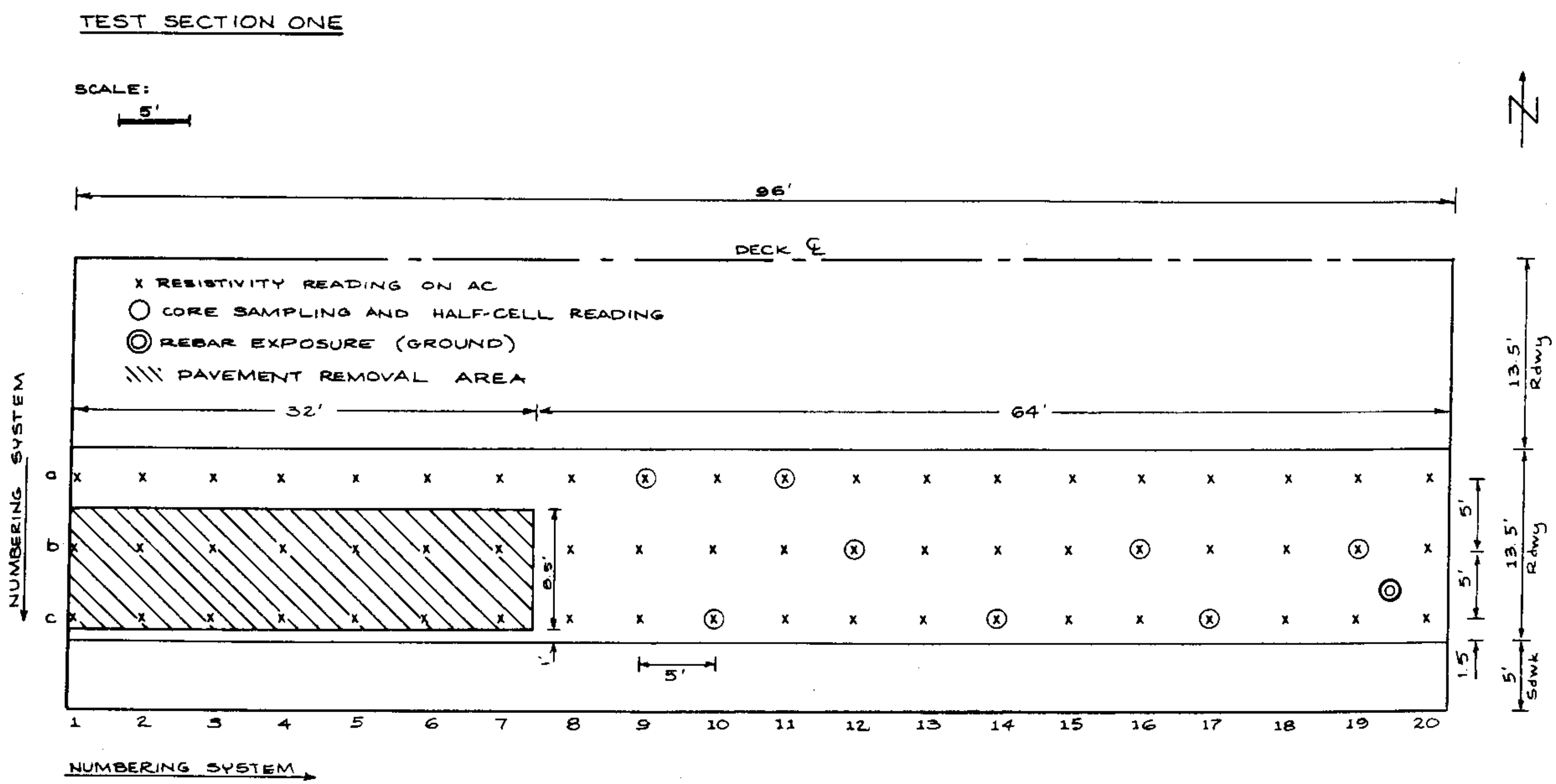

Figure 30. Reading Setup and Location of Pavement Removal Area (Columbia River Bridge Deck, test section 1)

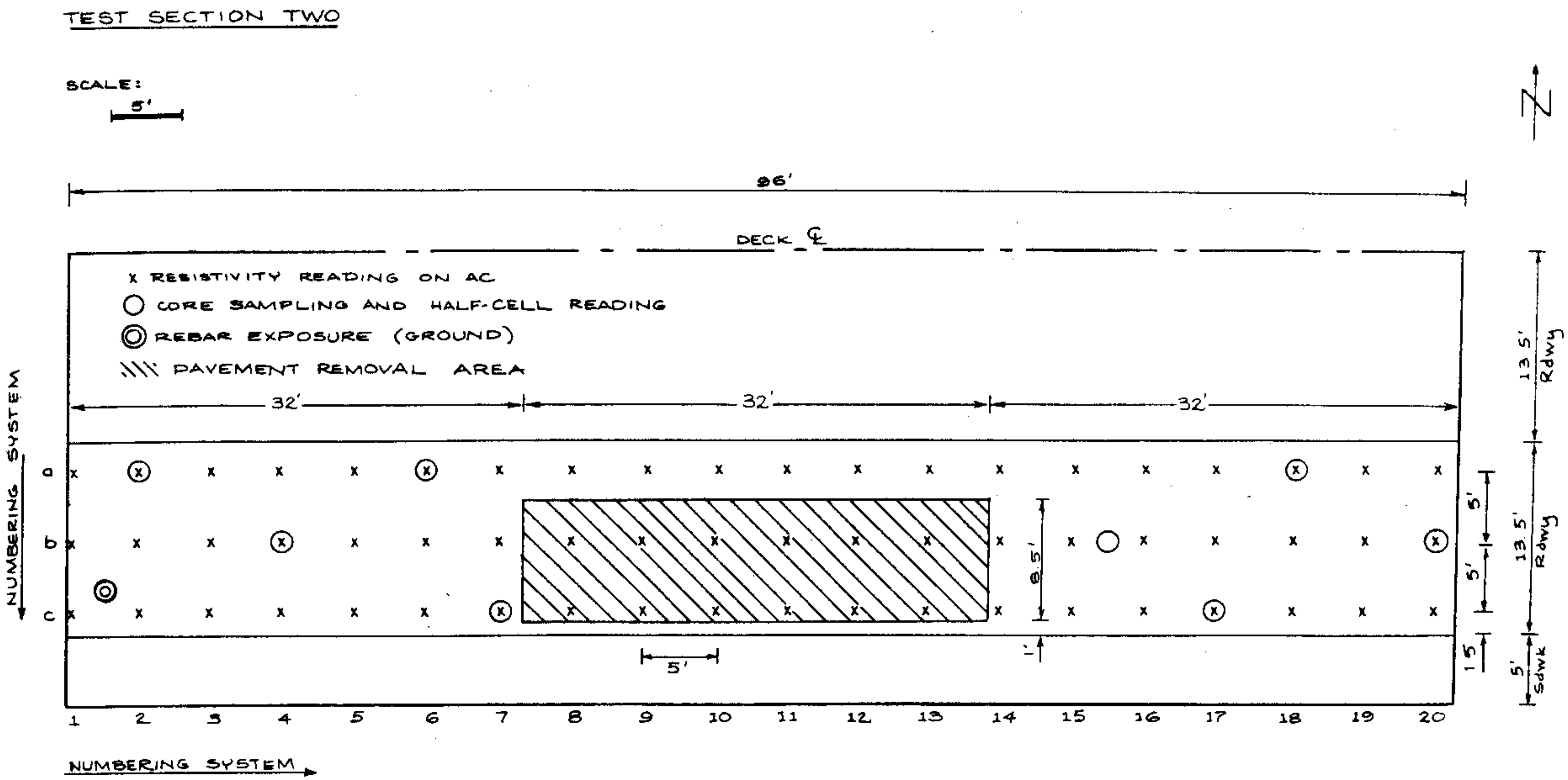

Figure 31. Reading Setup and Location of Pavement Removal Area
(Columbia River Bridge Deck, test section 2)

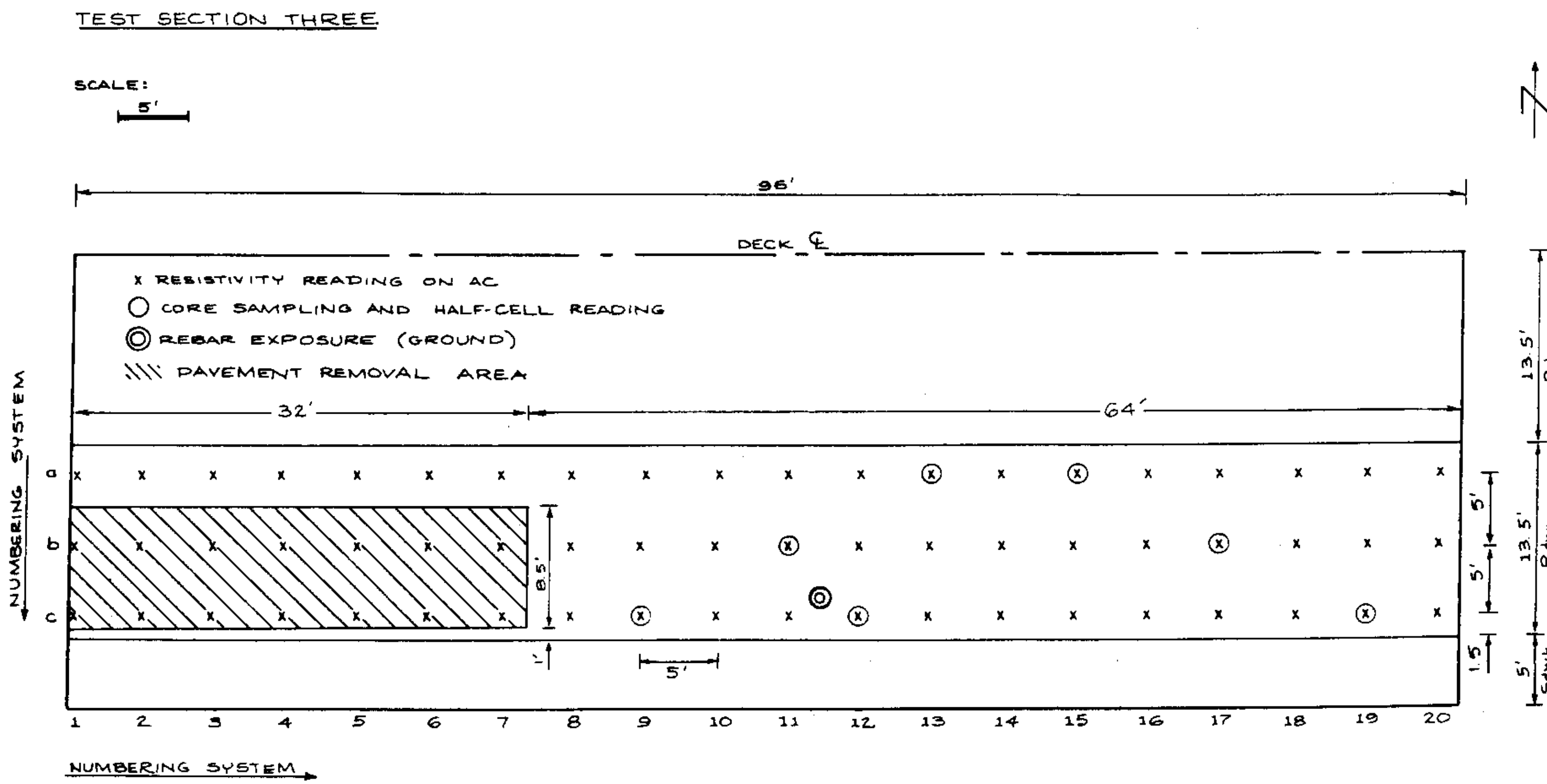

Figure 32. Reading Setup and Location of Pavement Removal Area (Columbia River Bridge Deck, test section 3)

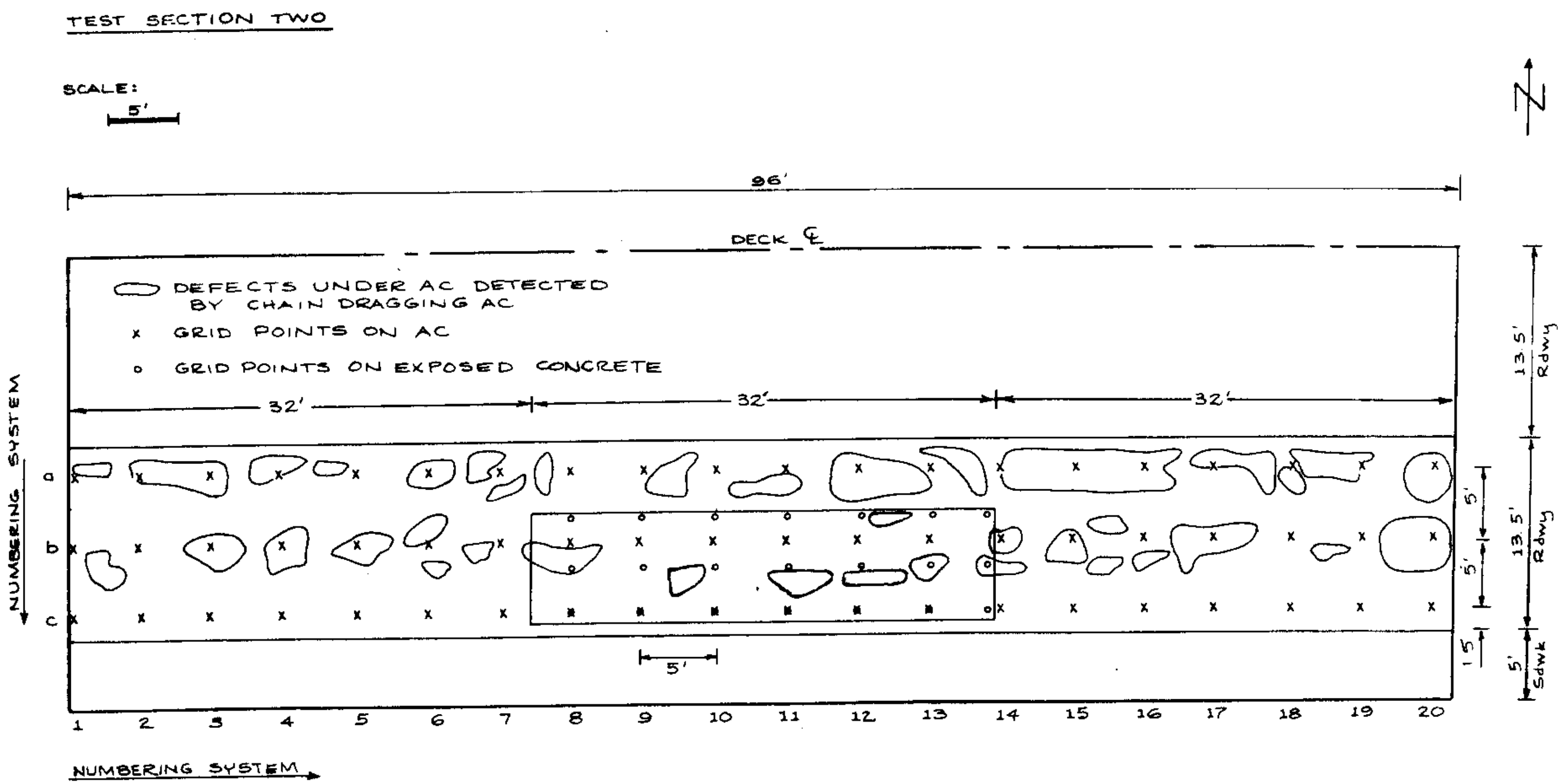

Figure 33. Defects Under Pavement Detected by Chain Dragging Asphalt Overlay (Columbia River Bridge Deck, test section 2)

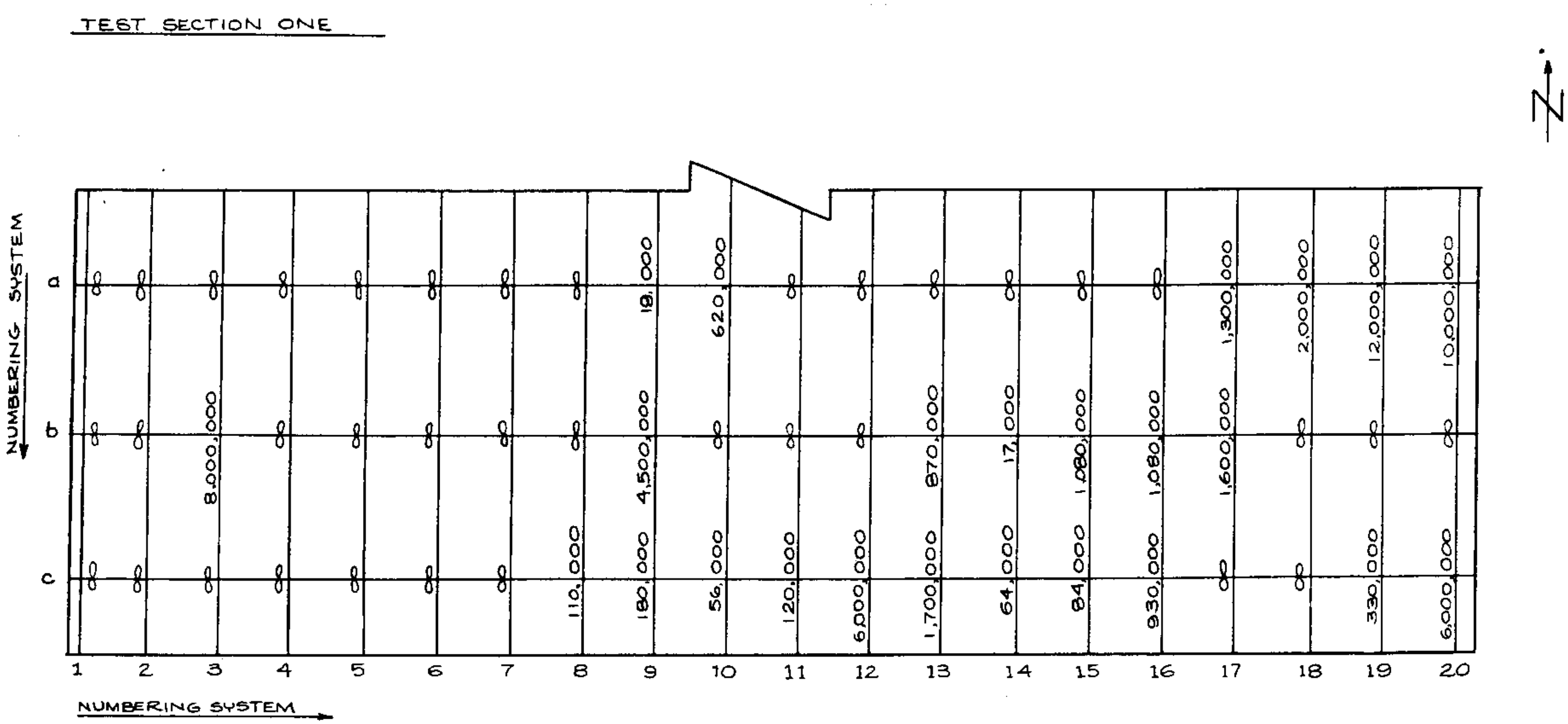

Figure 34. Results of Electrical Resistivity Testing on AC Overlay
(Columbia River Bridge Deck, test section 1)

Figure 35. Results of Electrical Resistivity Testing on AC Overlay (Columbia River Bridge Deck, test section 2)

TEST SECTION THREE

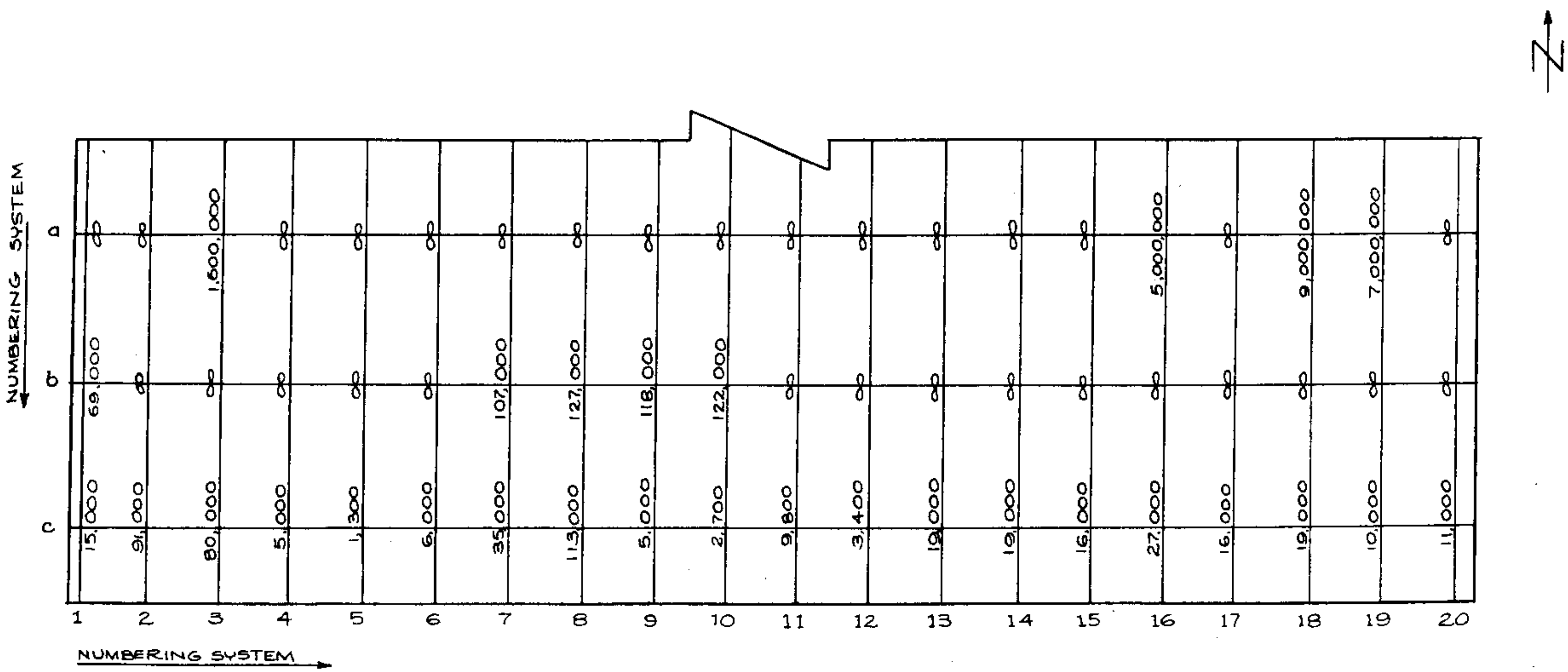

Figure 36. Results of Electrical Resistivity Testing on AC Overlay (Columbia River Bridge Deck, test section 3)

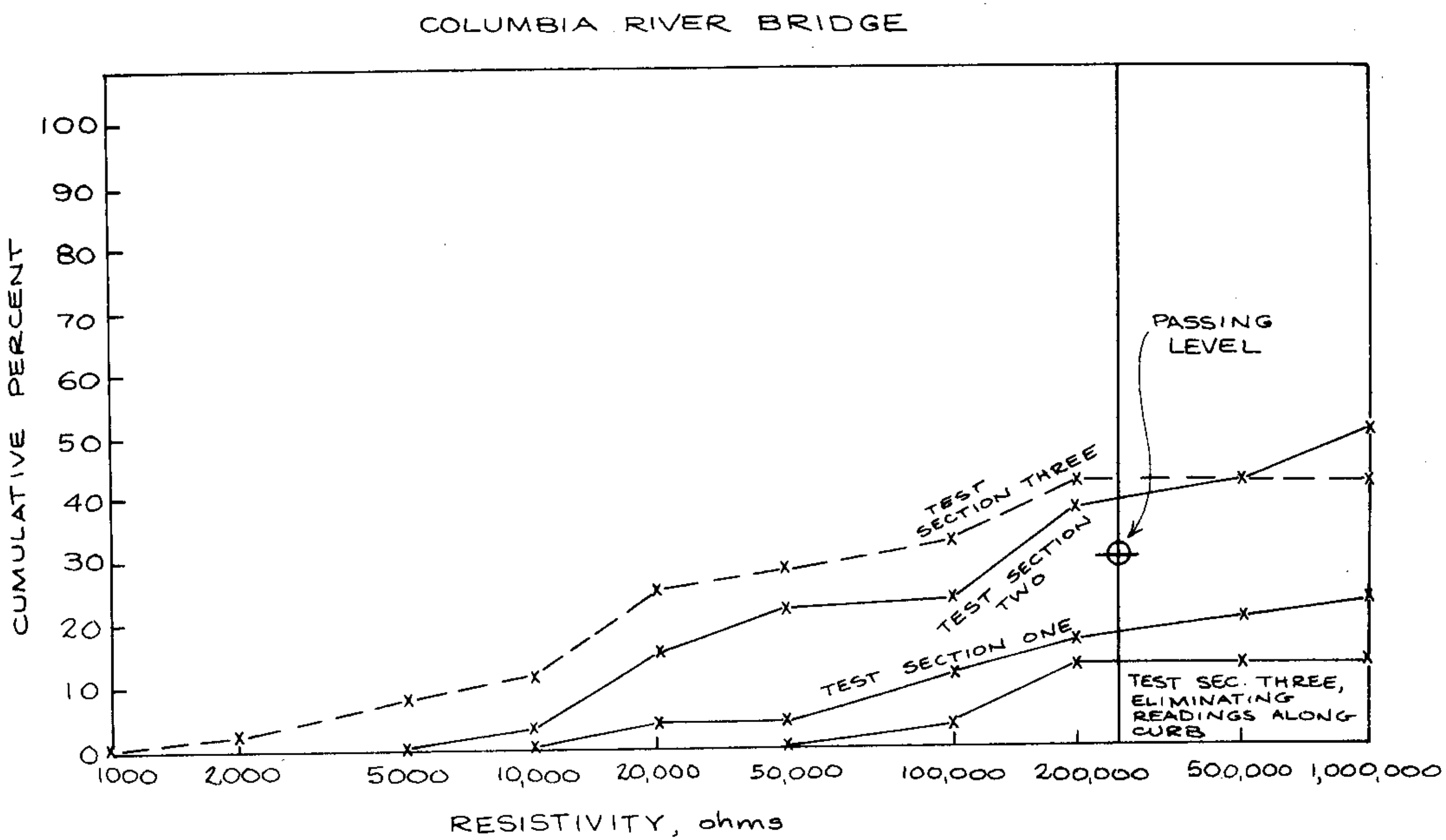

Figure 37. Electrical Resistivity Readings at Columbia River Bridge Deck

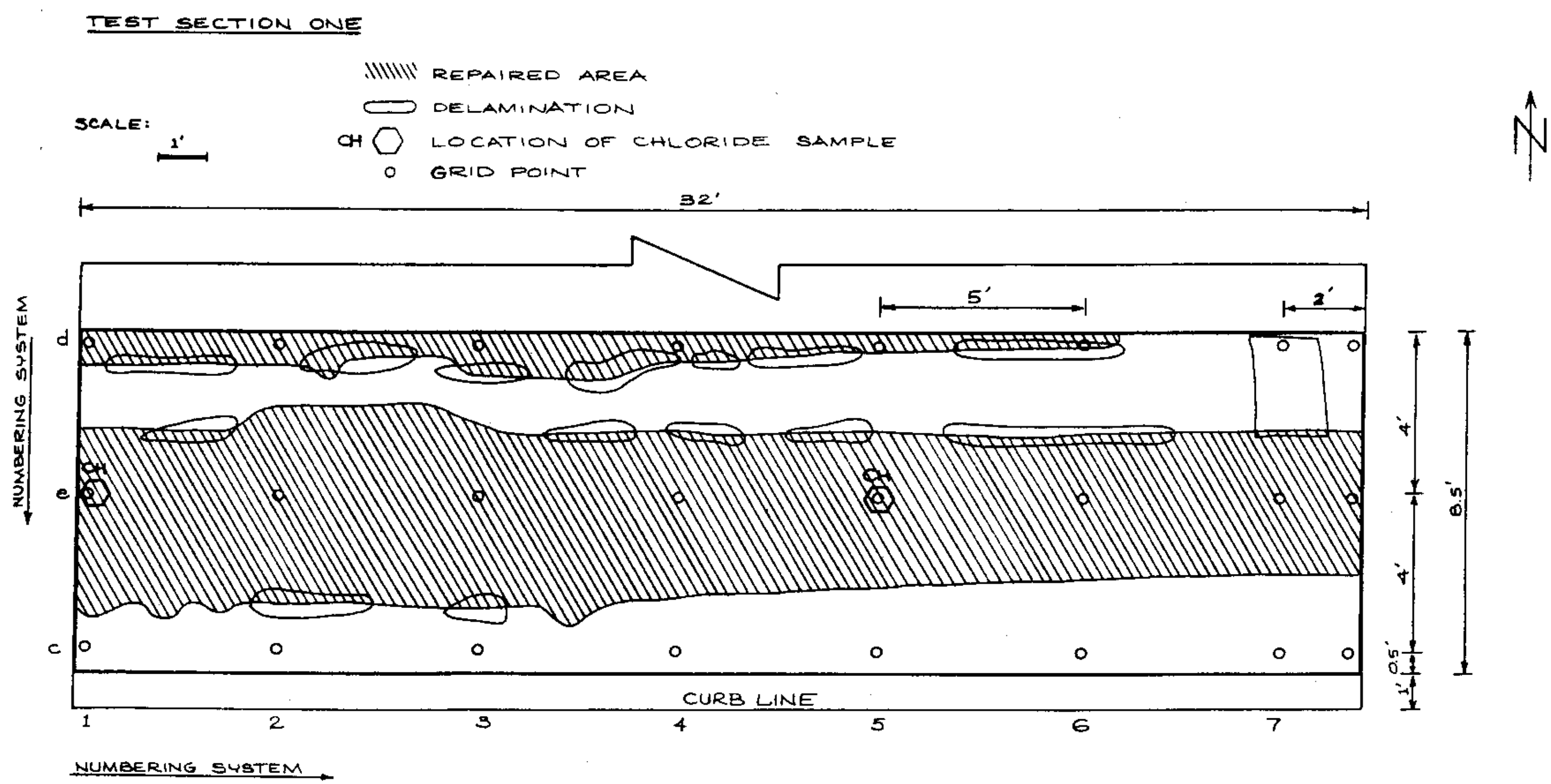

Figure 38. Repair Job and Defects on Exposed Concrete (Columbia River Bridge Deck, test section 1)

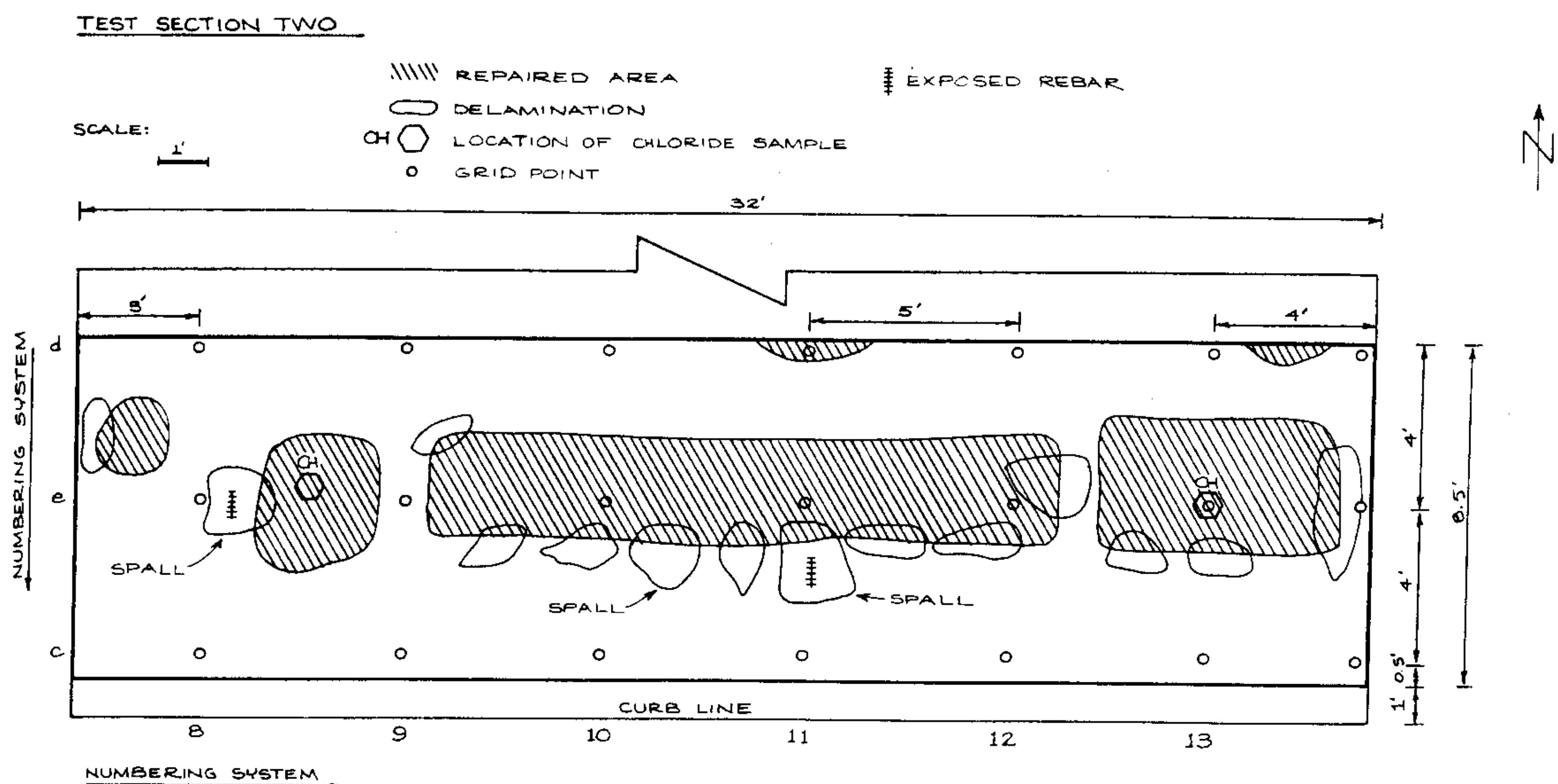

Figure 39. Repair Job and Defects on Exposed Concrete
(Columbia River Bridge Deck, test section 2)

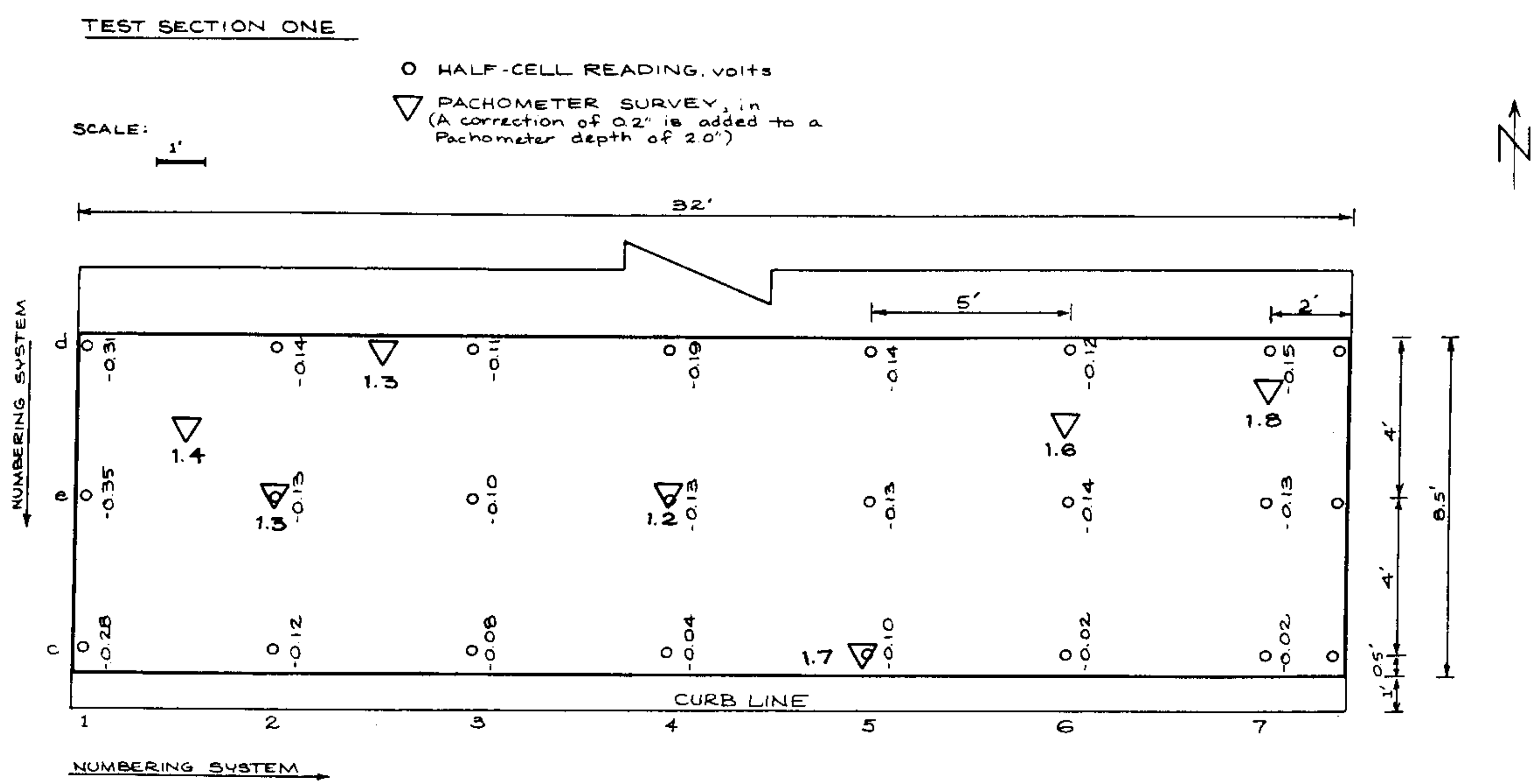

Figure 40. Reading Setup and Test Results on Exposed Concrete (Columbia River Bridge Deck, test section 1)

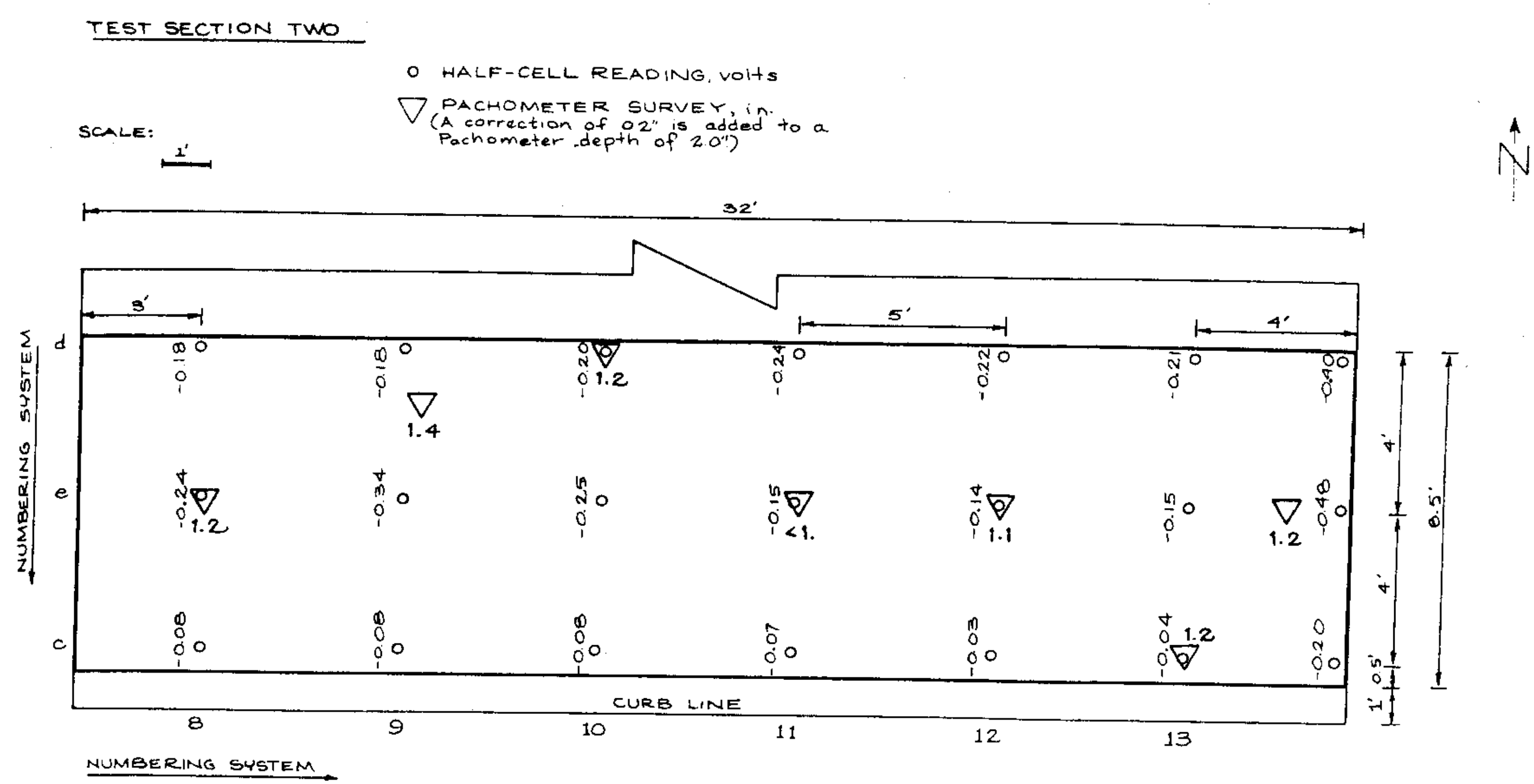

Figure 41. Reading Setup and Test Results on Exposed Concrete (Columbia River Bridge Deck, test section 2)

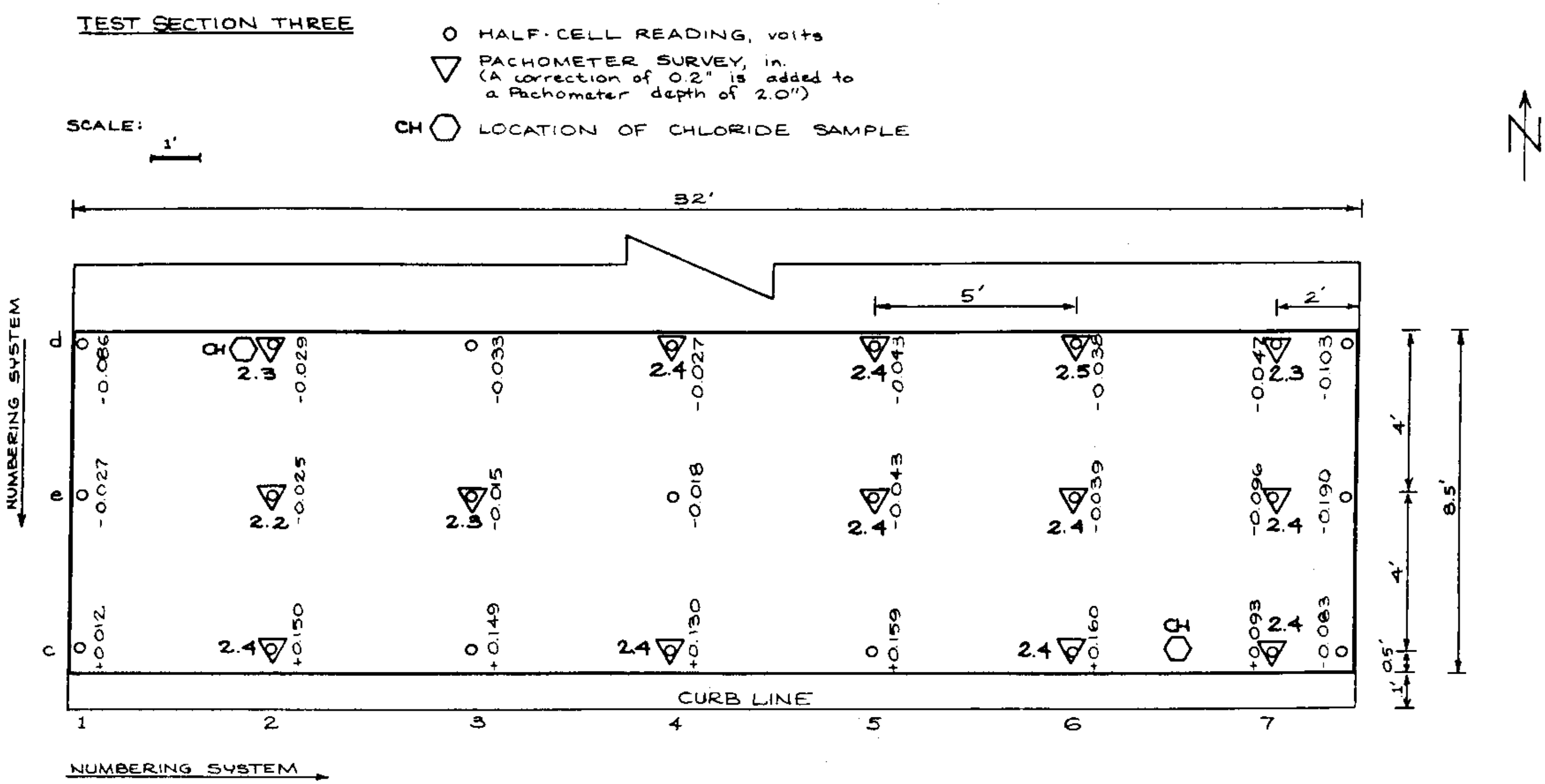

Figure 42. Reading Setup and Test Results on Exposed Concrete (Columbia River Bridge Deck, test section 3)

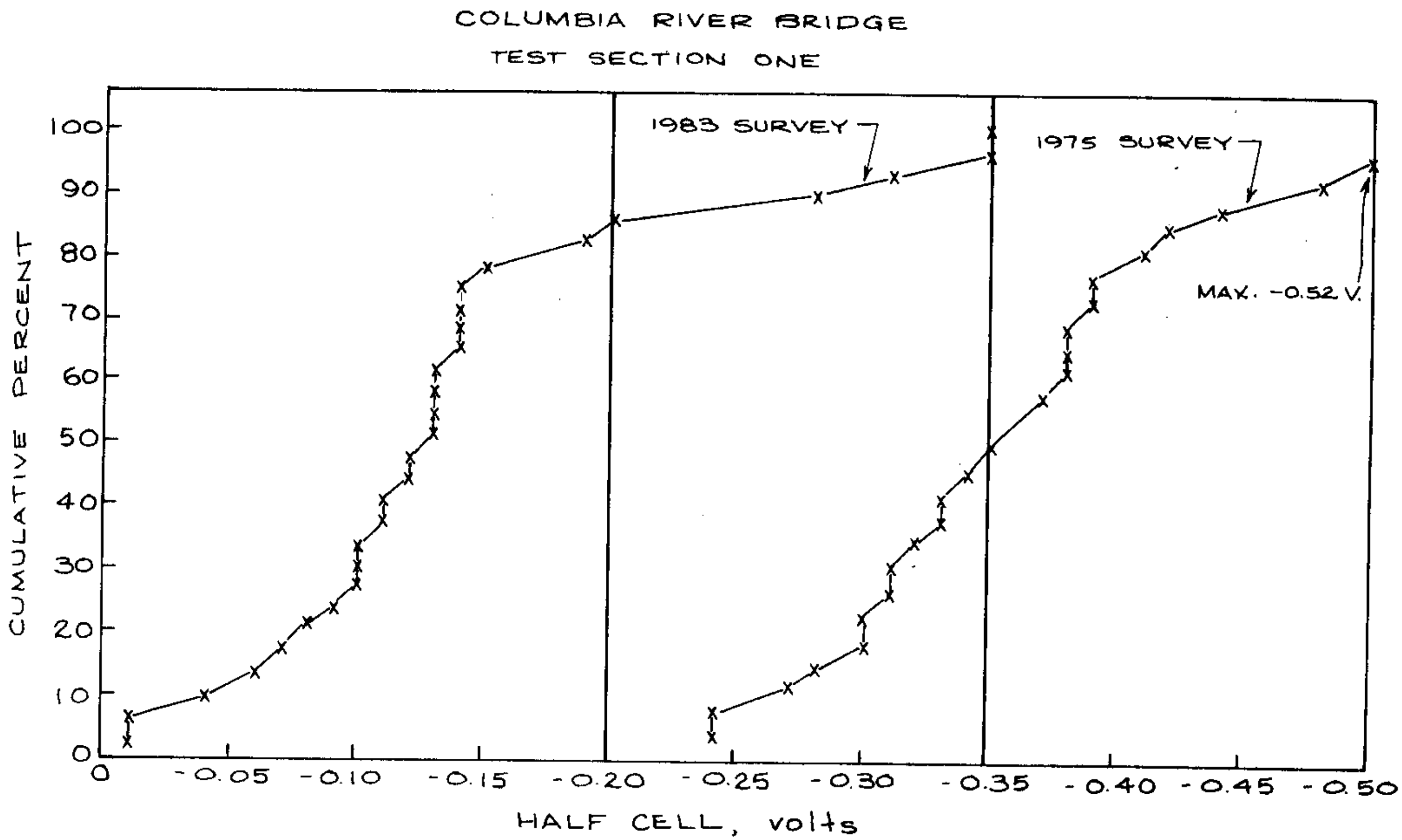

Figure 43. Half-Cell Readings for 1975 and 1983 (Columbia River Bridge Deck, test section 1)

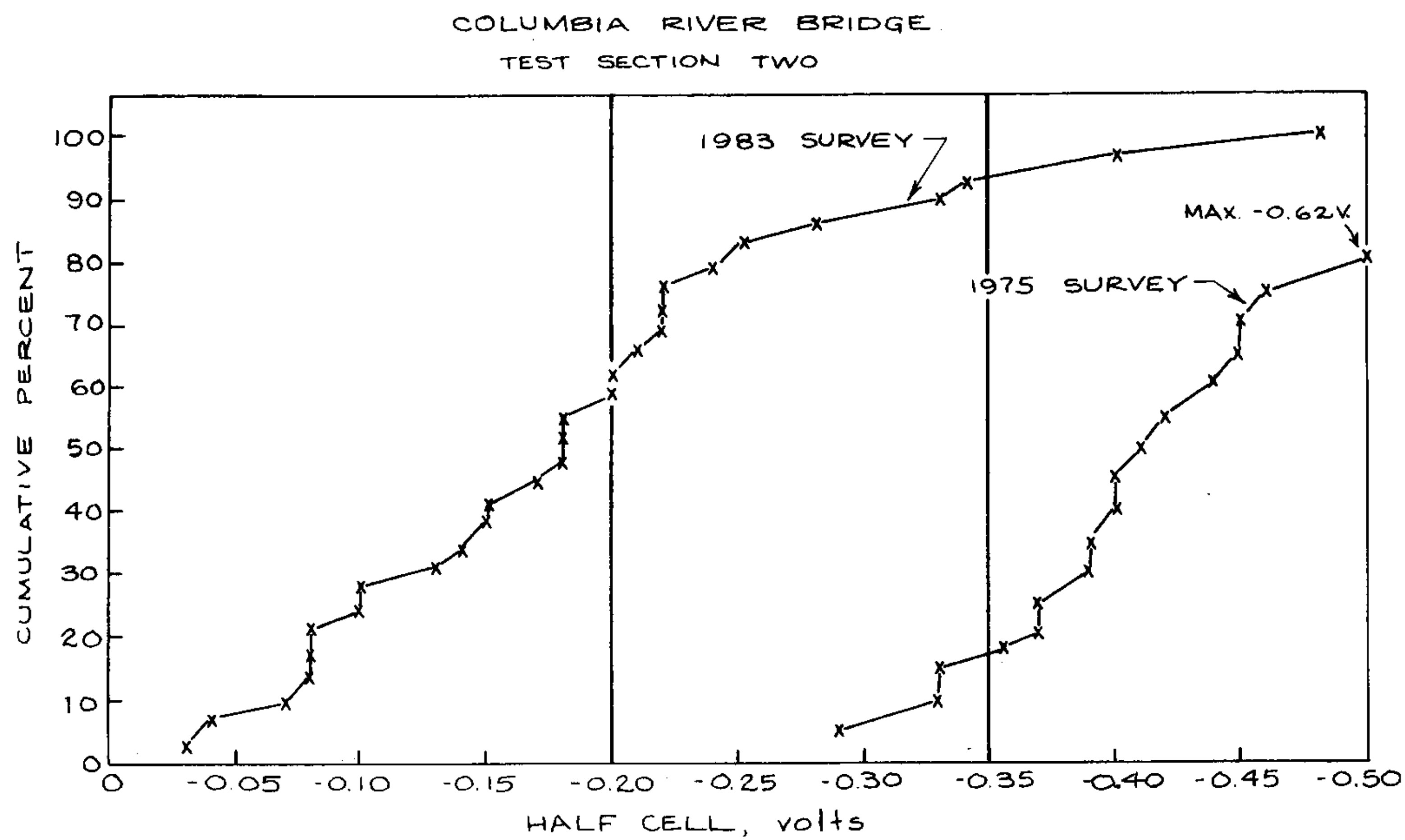

Figure 44. Half-Cell Readings for 1975 and 1983 (Columbia River Bridge Deck, test section 2)

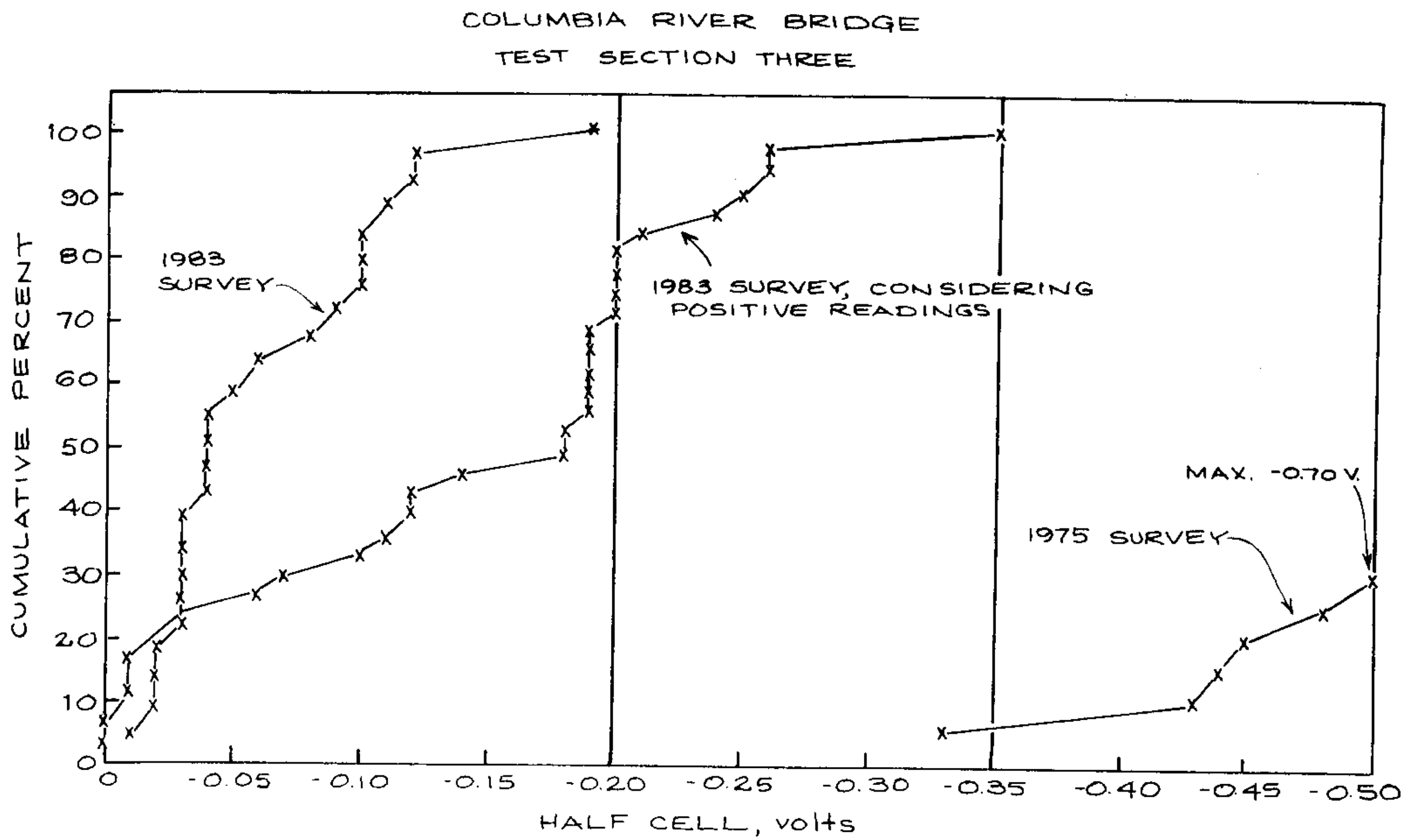

Figure 45. Half-Cell Readings for 1975 and 1983 (Columbia River Bridge Deck, test section 3)

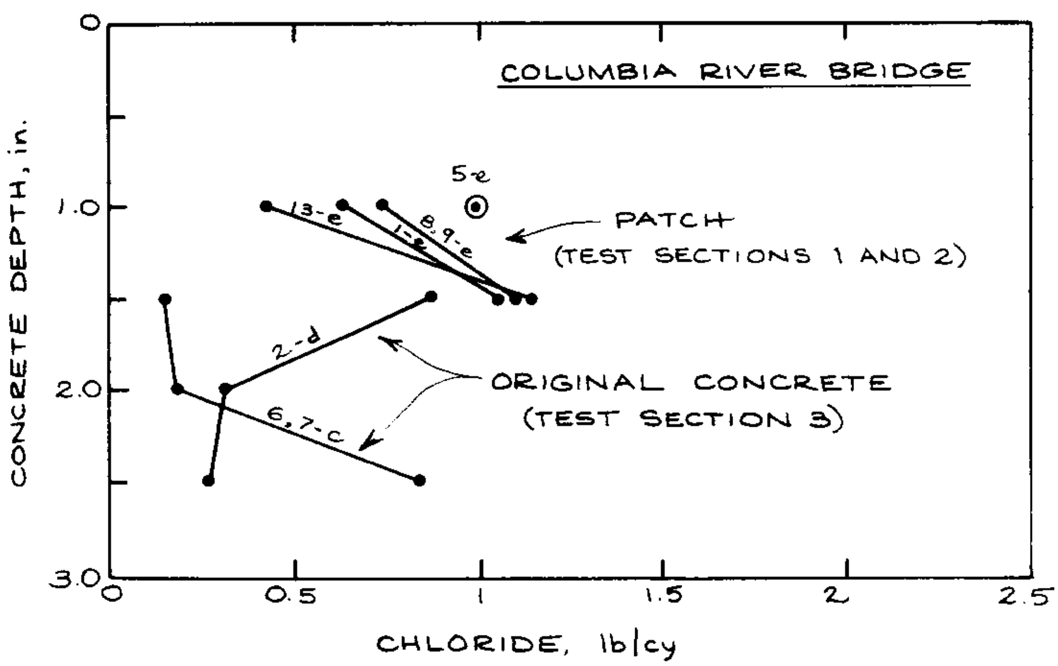

Figure 46. Chloride Content Versus Concrete Depth at Exposed Concrete Areas, Columbia River Bridge Deck

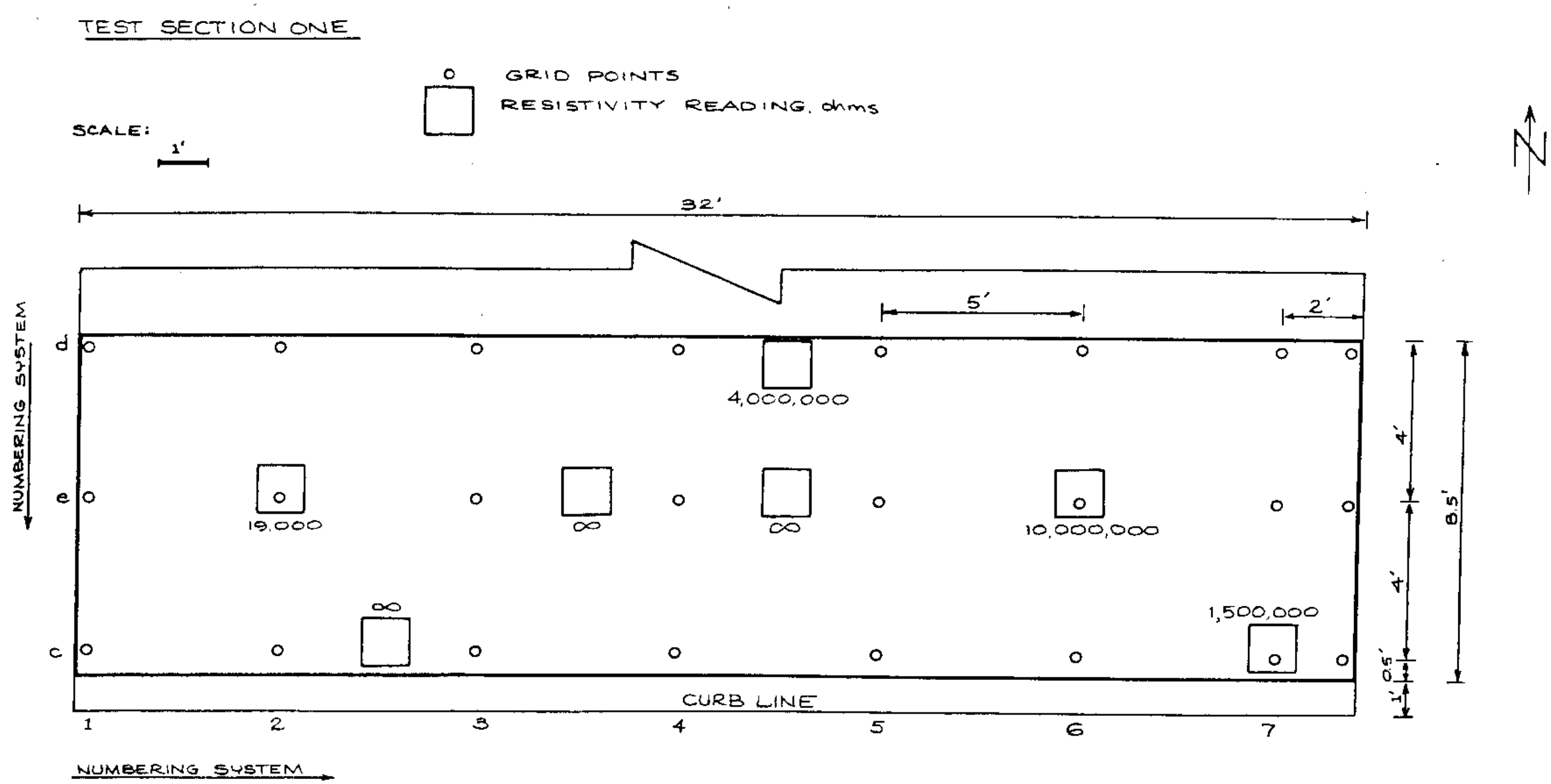

Figure 47. Electrical Resistivity Readings on Newly Installed Membrane (Columbia River Bridge Deck, test section 1)

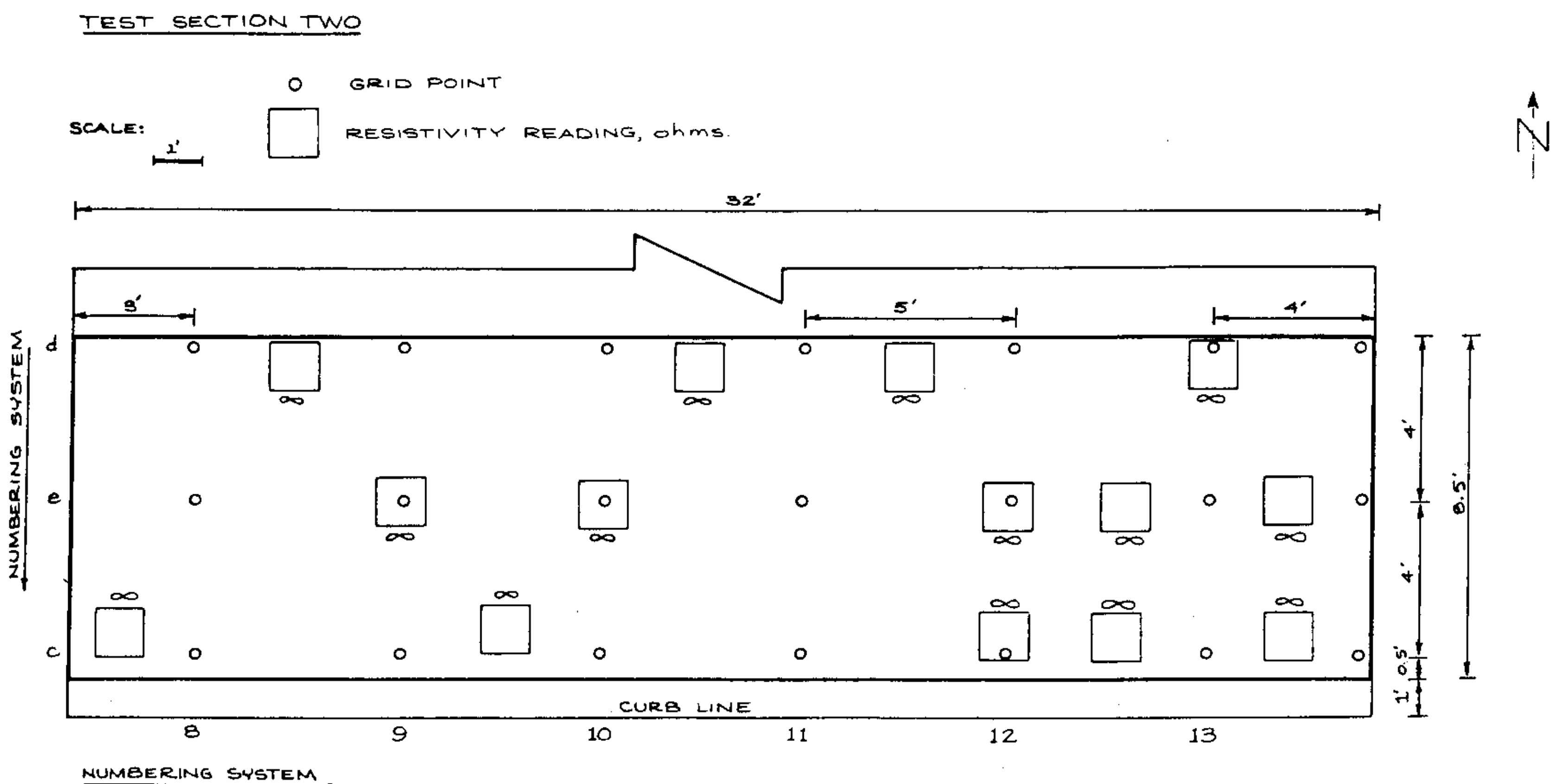

Figure 48. Electrical Resistivity Readings on Newly Installed Membrane (Columbia River Bridge Deck, test section 2)

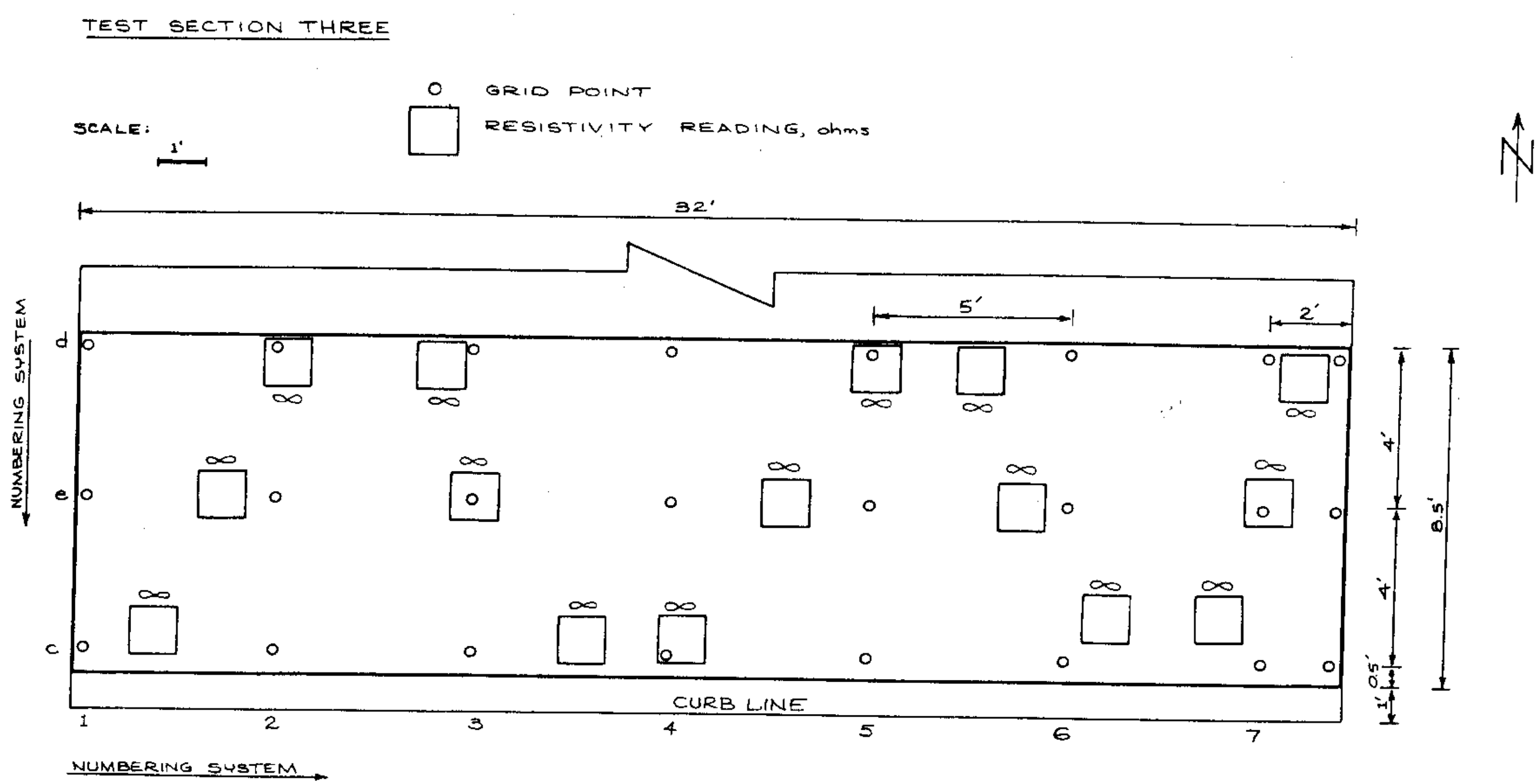

Figure 49. Electrical Resistivity Readings on Newly Installed Membrane (Columbia River Bridge Deck, test section 3)

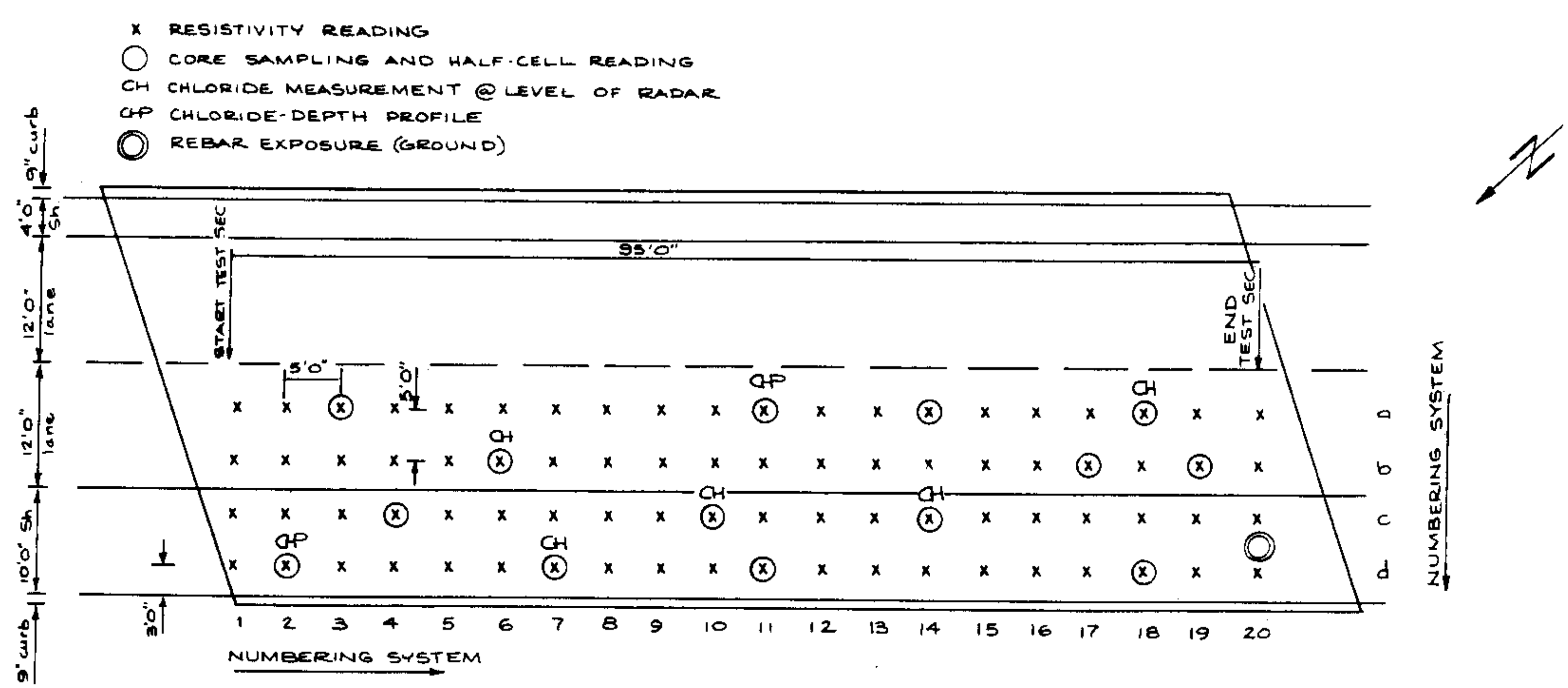

Figure 50. Test Section on Bridge 82/106 L/S (Roza Canal) and Test Reading Setup

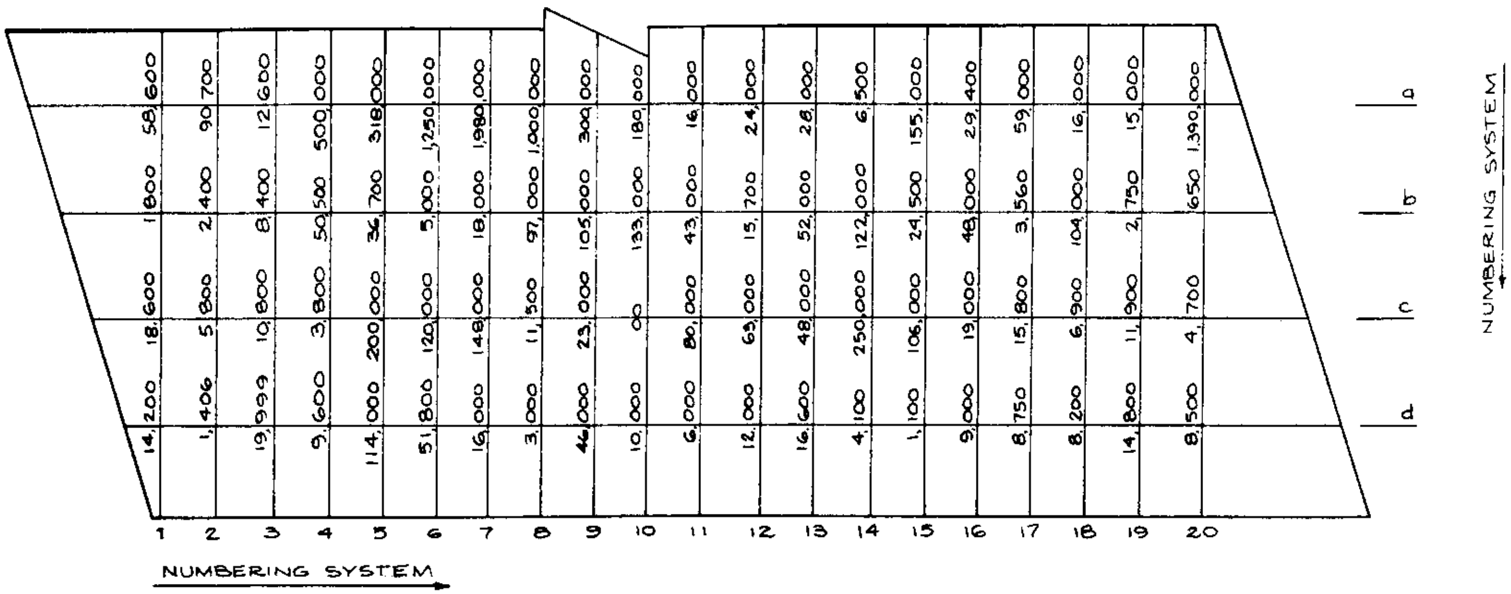

Figure 51. Results of Electrical Resistivity Testing on AC Overlay at Roza Canal Test Site

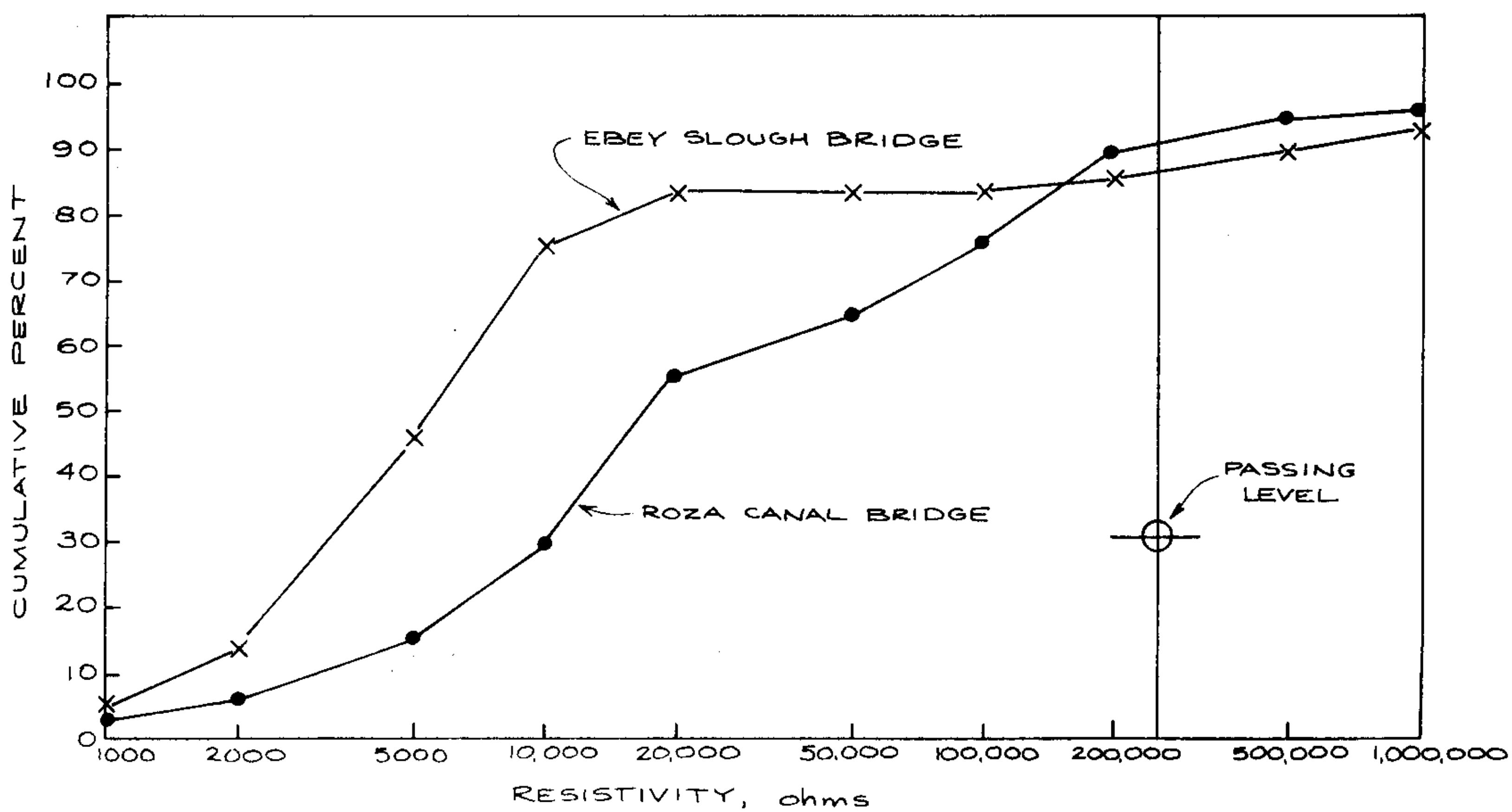

Figure 52. Electrical Resistivity Readings at Roza Canal and Ebey Slough Test Sites

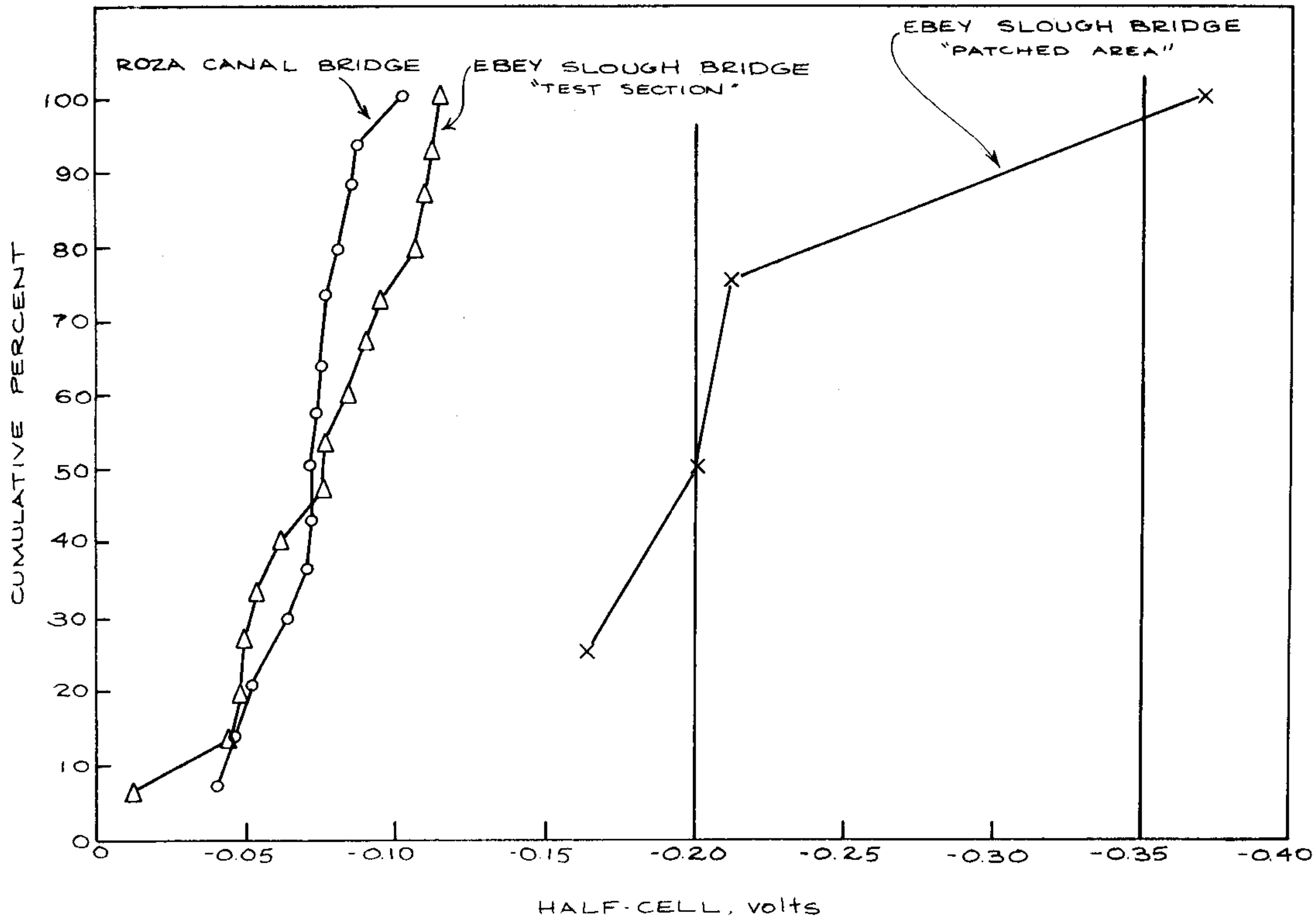

Figure 53. Electrical Potential Readings at Roza Canal and Ebey Slough Test Sites

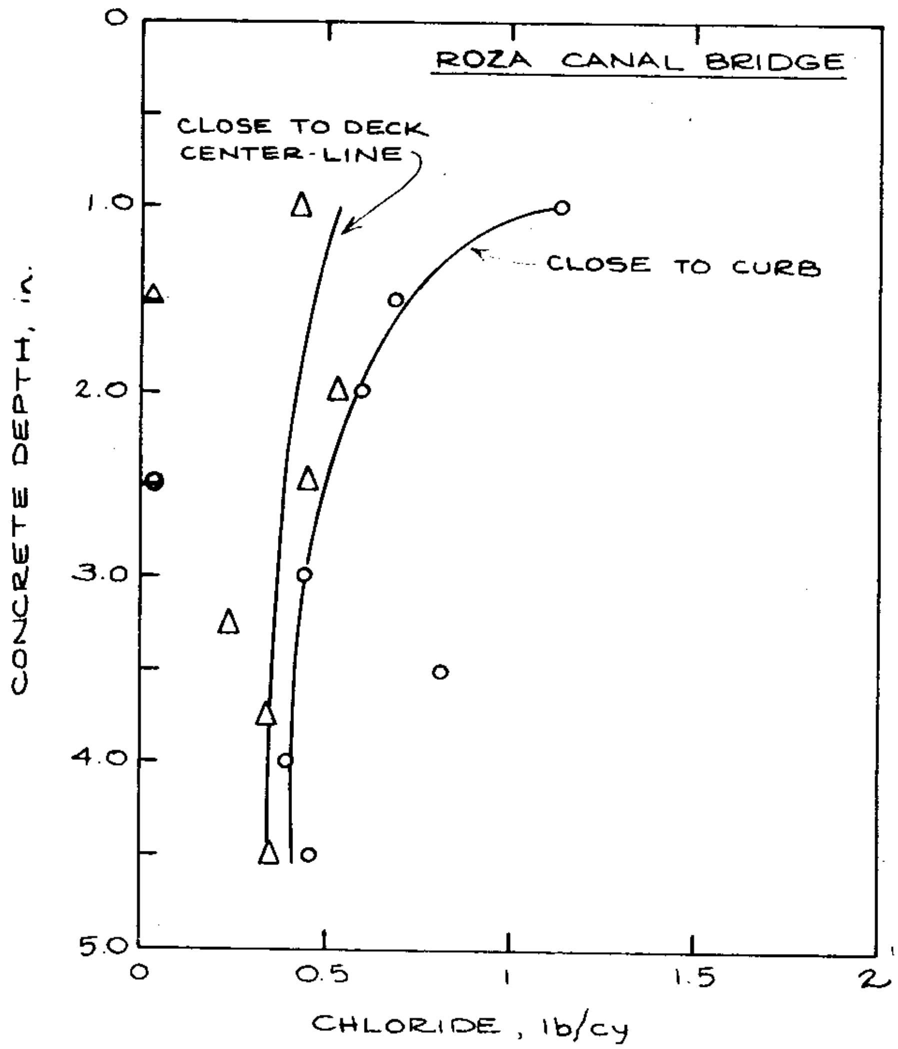

Figure 54. Chloride Content Versus Concrete Depth Profile at Roza Canal Test Site

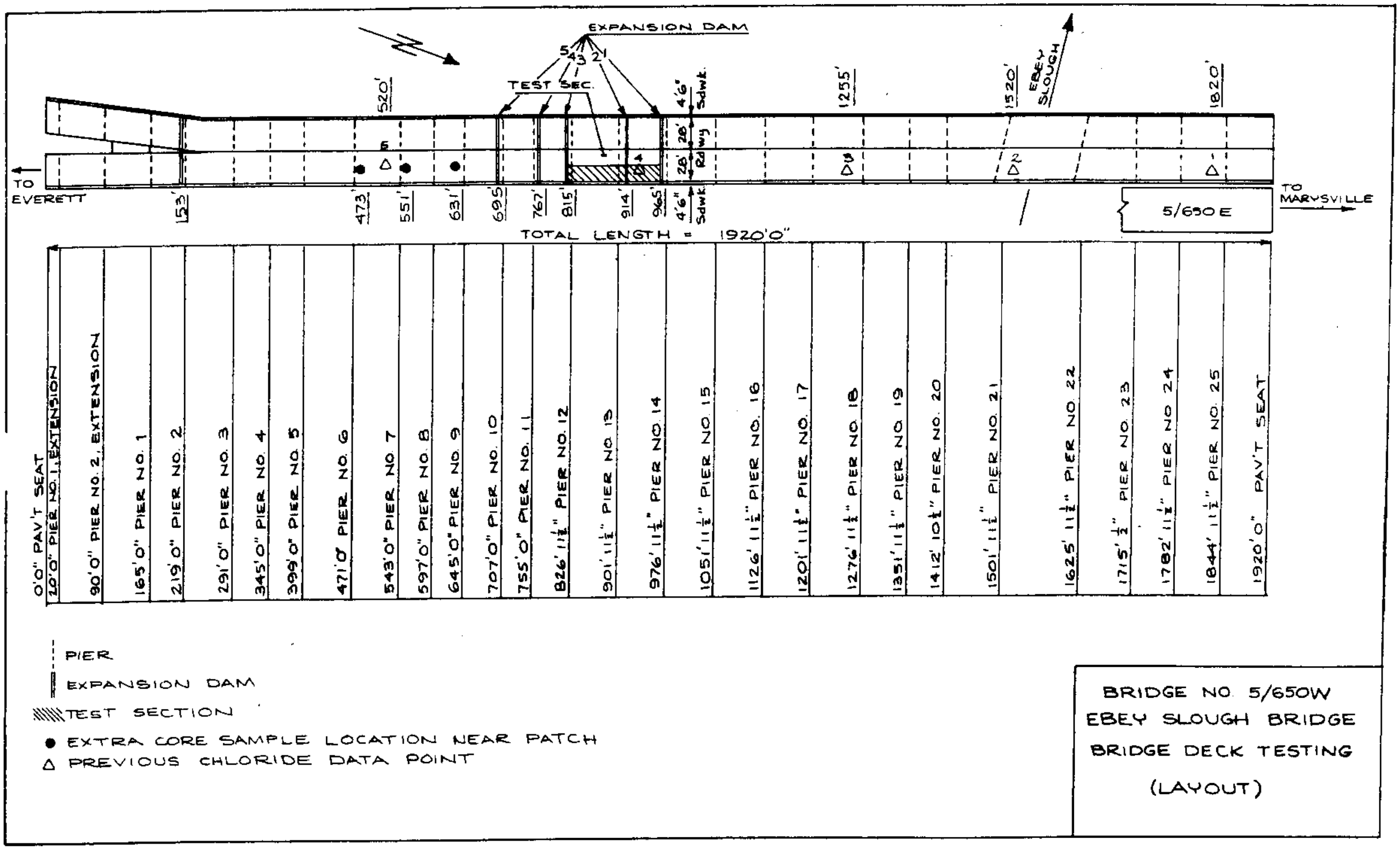

Figure 55. Location of Test Section and Extra Core Sampling at Ebey Slough Bridge Deck

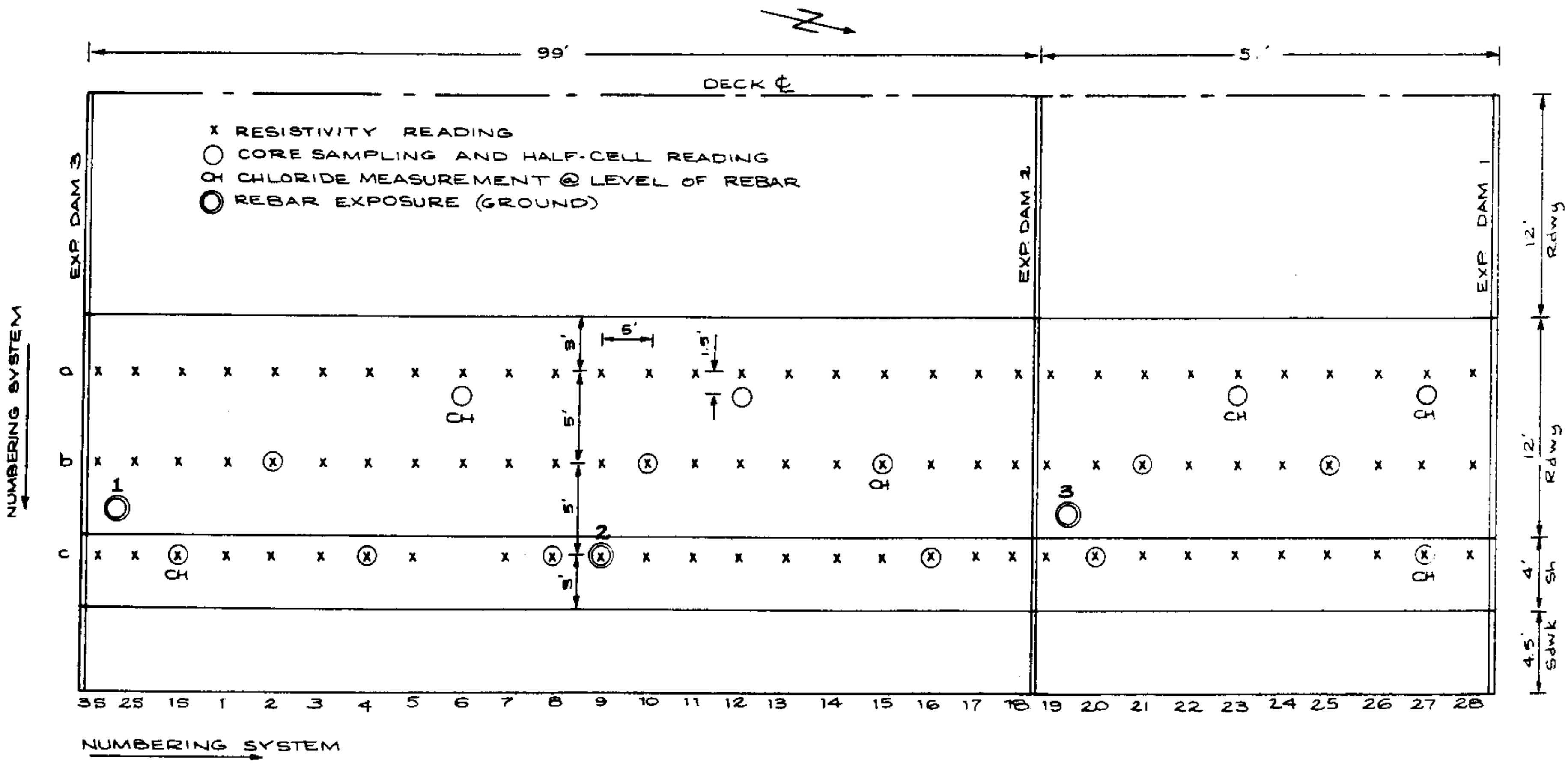

Figure 56. Test Reading Setup on Test Section of Ebey Slough Bridge

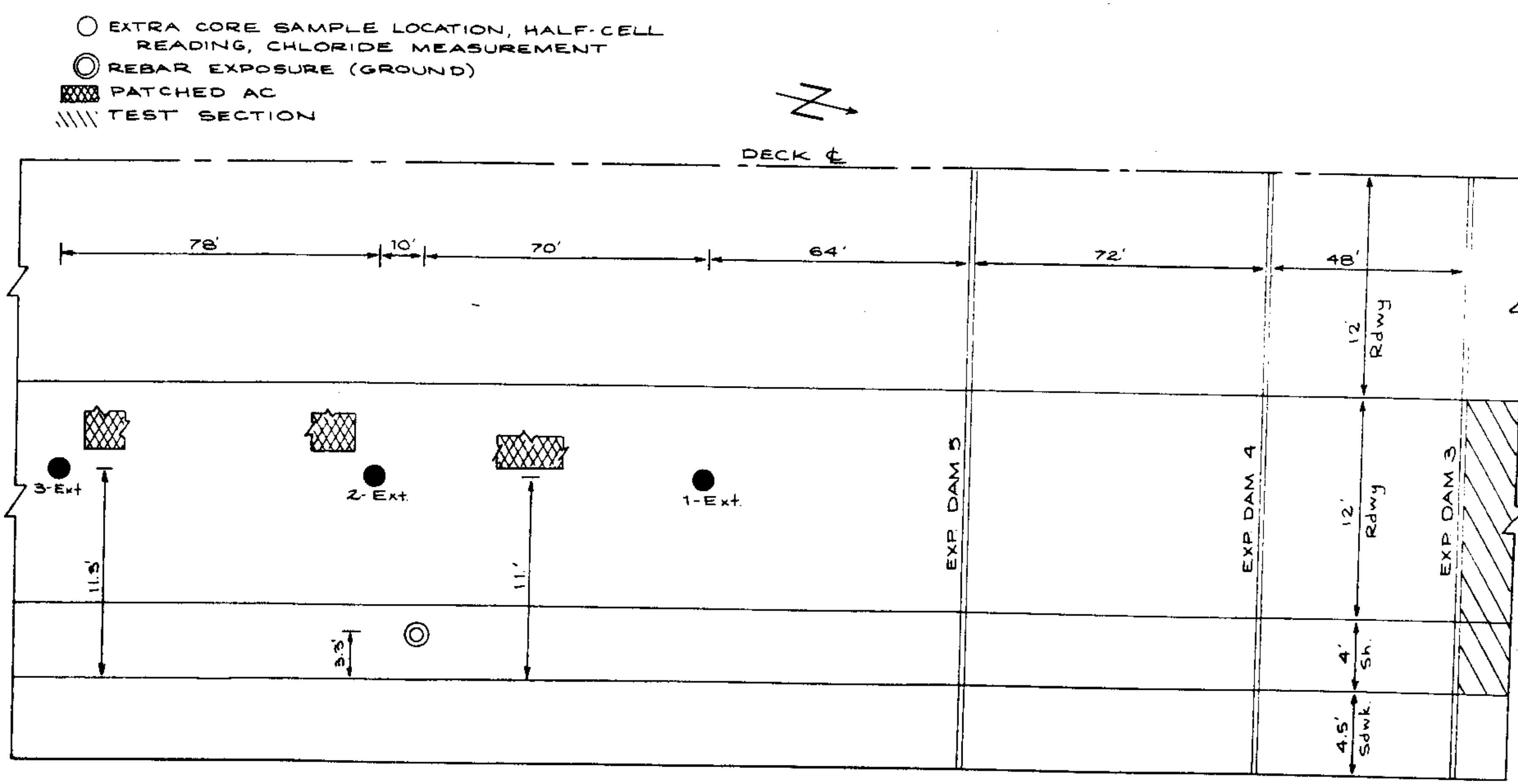

Figure 57. Details of Location of Extra Core Samples at Ebey Slough Bridge Deck

EXP DAM 3

NUMBERING SYSTEM	a	b	c
3S	18000	∞	∞
2S	3,070	10,480	240,000
1S	3,800	12,090	12,080
1	5,530	2,080	7,000
2	5,830	2,400	18,800
3	5,650	3,930	8,600
4	5,900	2,460	300,000
5	6,010	3,700	180,000
6	5,830	3,580	510,000
7	5,860	1,600	715,000
8	5,980	2,010	4,400
9	6,650	2,940	7,060
10	6,310	3,900	8,000
11	6,880	4,430	335,000
12	5,230	2,260	11,600
13	6,740	6,030	8,000
14	6,410	6,250	6,920
15	6,440	5,890	350
16	6,250	750	840
17	4,500	1,050	400
18	5,680	5,770	16,400
EXP DAM 2			
19	∞	1,750,000	6,300,000
20	4,370	2,640	500,000
21	4,600	2,100	1,850
22	5,020	1,850	2,000
23	4,450	1,710	4,070
24	4,920	2,270	2,970
25	4,760	2,710	1,800
26	4,330	1,380	1,440
27	3,200	2,260	∞
28	900	800,000	∞

EXP DAM 1

NUMBERING SYSTEM

Figure 58. Results of Electrical Resistivity Testing on AC Overlay at Ebey Slough Bridge Test Section

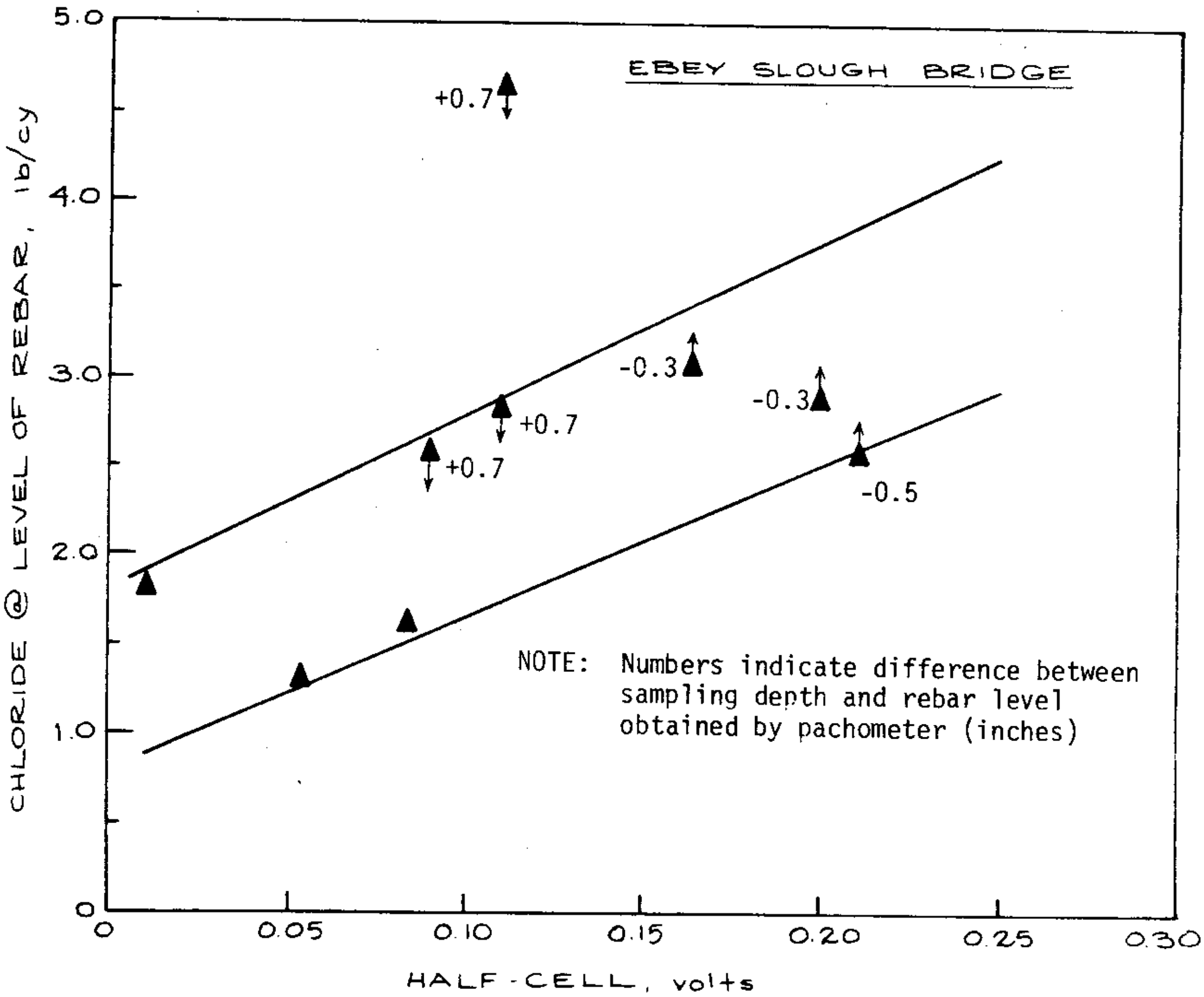

Figure 59. A Possible Correlation Between Chloride Content at the Level of Rebar and Corresponding Half-Cell Potentials

Table 1. Coring Program Data at Test Section 1, Columbia River Bridge Deck

Core No.	Membrane					Concrete Deck					Remarks
	AC + Membrane Thickness (in)[a]	Bond[b] to AC	Bond to Concrete	Pinholes[c]	Resistivity (ohms)	Concrete Surface		Total Cover (Pachometer) (in)	Concrete Cover (in)[d]	Half-Cell (volts)	
						Wetness	Soundness				
9-a	1.6	Good	Poor	--	19,000	Slight	Sound	2.6	1.9	-.350	
10-c	2.0	"	Good	--	56,000	None	"	2.8	1.7	-.095	
11-a	1.6	"	Fair	--	∞	"	"	2.4	1.6	-.141	
12-b	1.6	"	Fair	--	∞	"	"	2.4	1.6	-.092	
14-c	2.0	"	Good	--	64,000	"	"	2.9	1.7	-.067	
16-b	1.5	"	Good	--	1,080,000	"	"	2.5	1.8	-.105	
17-c	2.0	"	Good	--	∞	"	"	2.7	1.6	-.063	
19-b	1.9		Good	--	∞	"	"	2.4	1.3	-.201	
Ground	1.6		Fair	--	--	"	"	2.7	2.0	--	Actual total depth 3.6"

a: Actually measured
b: Integrity between AC overlay and membrane
c: Not visually detectable
d: Cover thickness = total thickness (pachometer) plus correction, minus AC and membrane thickness (a correction of 0.9" is considered for a total pachometer depth of 2.7"; see ground)

Table 2. Coring Program Data at Test Section 2, Columbia River Bridge Deck

Core No.	Membrane					Concrete Deck					Remarks
	AC + Membrane Thickness (in)[a]	Bond[b] to AC	Bond to Concrete	Pinholes[c]	Resistivity (ohms)	Concrete Surface: Wetness	Concrete Surface: Soundness	Total Cover (Pachometer) (in)	Concrete Cover (in)[d]	Half-Cell (volts)	
2-a	1.3	Good	Fair	--	∞	None	Sound	2.3	1.5	-.125	
4-b	1.4	"	Fair	--	16,000,000	None	"	2.5	1.7	-.097	
6-a	1.4	"	Poor	--	19,000	None	"	2.5	1.7	-.179	
7-c	2.0	"	Poor	--	25,000	Slight	"	2.7	1.3	-.100	
15,16-b	1.4	"	Poor	--	∞	None	"	2.5	1.7	-.216	
17-c	1.4	"	Fair	--	43,000	None		2.5	1.7	-.173	
18-a	1.5	"	Poor	--	1,600,000	None	"	2.3	1.3	-.283	
20-b	1.5	"	Fair	--	∞	None	"	2.3	1.3	-.325	
Ground	1.7	"	Fair	--	--	--	"	2.7	1.6	--	Actual total depth, 3.3"

a: Actually measured
b: Integrity between AC overlay and membrane
c: Not visually detectable
d: Cover thickness = total thickness (pachometer) plus correction minus AC and membrane thickness (a correction of 0.6" is considered for a total pachometer depth of 2.7", see ground)

Table 3. Coring Program Data at Test Section 3, Columbia River Bridge Deck

Core No.	Membrane					Concrete Deck					Remarks
	AC + Membrane Thickness (in)[a]	Bond[b] to AC	Bond to Concrete	Pinholes[c]	Resistivity (ohms)	Concrete Surface		Total Cover (Pachometer) (in)	Concrete Cover (in)[d]	Half-Cell (volts)	
						Wetness	Soundness				
9-c	2.0	Good	Fair	--	5,000	None	Sound	2.7	1.7	-.019	
11-b	1.6	"	Poor	--	∞	"	"	2.7	2.1	-.115	
12-c	1.6	"	Fair	--	3,400	"	"	2.9	2.4	-.008	
13-a	1.6	"	Good	--	∞	"	"	2.4	1.7	-.116	
15-a	1.6	"	Good	--	∞	"	"	2.4	1.7	-.101	
17-b	1.6	"	Fair	--	∞	"	"	2.8	2.3	-.110	
19-c	1.75	"	Poor	--	10,000	"	"	2.4	1.6	-.056	
Ground	1.8	"	Good	--	--	"	"	2.9	2.2	--	Actual total depth, 4"

a: Actually measured
b: Integrity between AC overlay and membrane
c: Not visually detectable
d: Cover thickness = total thickness (pachometer) plus correction minus AC and membrane thickness (a correction of 1.1" is considered for a total pachometer depth of 2.9"; see ground)

Table 4. Results of Chloride Content Measurement in 1975 and 1983, Columbia River Bridge Deck

Sample No.	Sample Location		Cl^- Measured in 1975 (lb/cy)	Cl^- Measured in 1983 (lb/cy)[a]	Increase in Cl^-		Average Increase in Cl^-	
	Distance from * (ft)	Distance from Curb (ft)			lb/cy	%	lb/cy	%
1	11	7	3.45	5.20	1.75	51	1.46	64
2	336	5	1.53	2.69	1.16	76		

*: Distance from east-end of the exposed concrete of test Section 3

a: Resamples of 1975 Survey, sampling depth in concrete, 1" to 1.5"

Table 5. Results of Chloride Content Measurement at the Exposed Concrete, Columbia River Bridge Deck

Test Section	Sample No.	Nature of Concrete	Depth in Concrete (in)	Cl^- (lb/cy)
1	1-e	Patch	½ - 1	0.62
			1 - 1½	1.05
	5-e	Patch	½ - 1	0.98
			1 - 1½	--
2	8,9-e	Patch	½ - 1	0.73
			1 - 1½	1.11
	13-e	Patch	½ - 1	0.42
			1 - 1½	1.14
3	2-d	Original	1 - 1½	0.86
			1½ - 2	0.31
			2 - 2½	0.26
	6,7-c	Original	1 - 1½	0.15
			1½ - 2	0.19
			2 - 2½	0.83

Table 6. Coring Program Data at Roza Canal Test Site

Core No.	Membrane					Concrete Deck						REMARKS
	AC + Membrane Thickness (in)[a]	Bond to AC	Bond to Concrete	Pinholes	Resistivity (ohms)	Concrete Surface		Total Cover (Pachometer) (in)	Concrete Cover (in)[b]	Cl^- at Level of Rebar (lb/cy)	Half-Cell (volts)	
						Wetness	Soundness					
2-d	0.9	Poor	Fair	None	1,406	Moderate	Sound	2.2	1.5	0.69	-.103	Actual total depth, 2.4"
3-a	0.75	Fair	Fair	Severe	12,600	Slight	"	2.4	2.0	--	-.047	
4-c	0.8	Fair	Poor	Moderate	3,800	Moderate	"	2.3	1.9	--	-.071	
6-b	1.0	Good	Fair	Severe	5,000	None	"	2.2	1.6	0.28	-.040	
7-d	0.8	Fair	Fair	None	16,000	Slight	"	2.0	1.6	0.31	-.051	
10-c	0.9	Fair	Poor	Moderate	∞	None	"	2.4	1.9	0.20	-.077	
11-a	0.9	Good	Good	Severe	16,000	None	"	2.2	1.7	0.54	-.086	
11-d	0.9	Poor	Poor	None	6,000	Severe	"	2.3	1.8	--	-.072	
14-a	0.9	Good	Fair	Severe	6,500	None	"	2.2	1.7	--	-.087	
14-c	0.8	Fair	Poor	Severe	250,000	None	"	2.3	1.9	0.41	-.074	
17-b	0.9	Poor	Poor	Severe	3,560	Moderate	"	2.3	1.8	--	-.072	
18-a	0.8	Good	Good	None	16,000	None	"	2.2	1.8	0.29	-.063	
13-d	0.9	Fair	--	Slight	8,200	Slight	"	2.3	1.8	--	-.080	
19-b	0.8	Poor	Fair	Moderate	2,750	None	"	2.3	1.9	--	-.075	
Ground	1.25	Good	Fair	Moderate	--	Moderate	"	2.4	1.75	--	--	Actual total depth, 3"

a: Actually measured.
b: Cover thickness = total thickness (pachometer) plus correction minus AC and membrane thickness (an average correction of 0.4" is considered for any total pachometer depth; see samples 2-d and ground)

Table 7. Chloride Content Versus Depth at Roza Canal Test Site

Sample 2-d		Sample 11-a	
Concrete Depth (in)	Cl^- (lb/cy)	Concrete Depth (in)	Cl^- (lb/cy)
1/2 - 1	1.13	1/2 - 1	0.43
1 - 1 1/2	0.69	1 - 1 1/2	0.00
1 1/2 - 2	0.59	1 1/2 - 2	0.54
2 - 2 1/2	0.02	2 - 2 1/2	0.45
2 1/2 - 3	0.44	2 1/2 - 3 1/4	0.24
3 - 3 1/2	0.81	3 1/4 - 3 3/4	0.34
3 1/2 - 4	0.39	3 3/4 - 4 1/2	0.35
4 - 4 1/2	0.46	-	-

Table 8. Coring Program Data of Ebey Slough Bridge Deck

Test Section

Core No.	Membrane: AC + Membrane Thickness (in)[a]	Membrane: Bond to AC	Membrane: Bond to Concrete	Membrane: Pinholes	Membrane: Resistivity (ohms)	Concrete Deck: Concrete Surface: Wetness	Concrete Deck: Concrete Surface: Soundness	Concrete Deck: Total Cover (Pachometer) (in)	Concrete Deck: Concrete Cover (in)[c]	Concrete Deck: Cl^- at Level of Rebar (lb/cy)	Concrete Deck: Half-Cell (volts)	REMARKS
15-c	1.5[b]	--	--	--	12,080	--	--	--	2.0[d]	1.46	-.084	
2-b	" [b]	Fair	Fair	None	2,400	Moderate	Scaling	--	" [d]	--	-.076	
4-c	" [b]	Fair	Fair	Slight	300,000	Slight	Sound	--	" [d]	--	-.063	
6-a	" [b]	Fair	Fair	None	5,330	Slight	"	--	" [d]	1.30	-.053	
8-c	" [b]	Fair	Poor	Severe	4,400	Moderate	"	--	" [d]	--	-.048	
10-b	" [b]	Fair	Fair	Moderate	3,900	None	"	--	" [d]	--	-.049	
12-a	" [b]	Good	Good	Severe	11,600	None	"	--	" [d]	--	-.045	
15-b	" [b]	Fair	Poor	None	5,890	Severe	"	--	" [d]	1.80	-.011	
16-c	" [b]	Fair	Fair	--	840	Moderate	Scaling	--	" [d]	--	-.114	
20-c	1.6	Poor	Poor	Severe	500,000	Severe	Sound	2.8	1.9	--	-.107	Membrane overlapped
21-b	1.4	Good	Good	Slight	2,100	Slight	"	3.0	2.3	--	-.095	
23-a	1.3	Fair	Fair	Moderate	4,450	Slight	"	2.6	1.9	<2.58	-.090	Cl^- 0.4" above rebar
25-b	1.3	Poor	Poor	None	2,710	Severe	"	2.7	2.0	--	-.076	
27-a	1.3	Good	Good	None	3,200	None	"	2.8	2.3	<2.82	-.110	Cl^- 0.7" above rebar
27-b	1.4	Good	Fair	Slight	2,260	None	"	2.8	2.2	<4.65	.112	Cl^- 0.7" above rebar
Ground 1	2.0	Fair	Poor	Severe	--	Slight	"	2.8	1.5	--	--	
Ground 2	1.7	Poor	Poor	Slight	--	Slight	"	--	2.0	--	--	Actual depth measured
Ground 3	1.5	Fair	Poor	Severe	--	Slight	"	--	2.0	--	--	Actual depth measured

continued

a: Actually measured
b: Average thickness in test section
c: cover thickness = total thickness (pachometer) plus correction minus AC and membrane thickness (a correction of 0.7" is considered for a total pachometer depth of 2.8"; comparison of sample 20-c and Ground 3).
d: Average cover thickness in test section

Table 8 (continued)

	Core No.	Membrane					Concrete Deck							REMARKS
		AC + Membrane Thickness (in)[a]	Bond to AC	Bond to Concrete	Pinholes	Resistivity (ohms)	Concrete Surface		Total Cover (Pacho-meter) (in)	Concrete Cover (in)[c]	Cl^- at Level of Rebar (lb/cy)	Half-Cell (volts)		
							Wetness	Soundness						
Near AC Patches	1-Ext.	1.4	Poor	Poor	--	--	Slight	Sound	2.0	1.1	>2.52	-.212		Cl^- 0.5" below rebar
	2-Ext.	1.0	Poor	Poor	Moderate	--	Moderate	Sound	2.0	1.5	>3.06	-.164		Cl^- 0.3" below rebar
	3-Ext.	1.0	Poor	Poor	Moderate	--	Slight	Delaminated	2.0	1.5	>2.85	-.200		Cl^- 0.3" below rebar
	AC Patch	--	--	--	--	--	--	--	--	--	--	-.370		
	Ground	--	Fair	Good	Moderate	--	None	Sound	--	--	--	--		

Table 9. Results of Chloride Content Measurement in 1974 and 1983 on Ebey Slough Bridge Deck

Sample No.	Sample Location		Cl^- Measured in 1974 (lb/cy)	Cl^- Measured in 1983 (lb/cy)[a]	Increase in Cl^-		Average Increase in Cl^-	
	Distance from North Pavement Seat (ft)	Distance from Curb (ft)			lb/cy	%	lb/cy	%
1	100	10.0	0.66	0.75	0.09	14	0.59	33
2	400	7.5	1.08	1.40	0.32	30		
3	665	8.5	1.28	0.14[b]	--	--		
4	990	16.0	2.31	>2.58[c]	>0.27	>12		
5	1400	12.5	2.24	<3.90[d]	<1.66	<74		

a: Resamples of 1974 survey, sampling depth in concrete 0.50 in to 1.25 in

b: Erratic result

c: Sample 23-a, measured at 0.95 in to 1.45 in

d: Sampling depth 0.50 in to 1.00 in