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FINAL REPORT

TWO-COURSE BONDED CONCRETE BRIDGE DECK CONSTRUCTION

"Condition and Performance After Six Years"

by

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

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

Charlottesville, Virginia

May 1981
VHTRC 81-R50

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SUMMARY

This report presents the findings from a six-year study of two-course bonded concrete bridge decks constructed in Virginia. Each of three special portland cement concretes was applied as an overlay, or wearing course, on two experimental spans. The overlays were a latex-modified, a low-water/cement and a wire-fiber concrete. Two spans constructed by a conventional single-lift technique on nearby structures with ordinary concrete served as controls for the study.

The report summarizes the evaluation of the construction, concrete properties, condition, and performance of the eight study spans through 1980. The condition and performance of the study spans warrant the use of two-course bonded bridge deck construction in four primary applications cited in the recommendations of the report.

The latex-modified and low-w/c concretes exhibited improved resistance to chloride ion penetration as compared to ordinary concrete, but the wire-fiber concrete did not. Suggestions are made conderning a quality assurance program for the latex-modified concrete and a program for monitoring such installations to determine the particular conditions that may predispose them to cracking.

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INTRODUCTION

The two-course bonded technique for constructing concrete bridge decks is merely a modification of the construction procedure for a normally designed deck cast in a single lift. The two-course deck is placed in two lifts, the first lift being struck off just above the level of the topmost reinforcing bars. Subsequent to hardening of the first lift, a second lift, or overlay, is placed to serve as protective cover for the reinforcing steel and as the wearing course for the deck.

The two-course bonded technique was recommended in 1974 by the American Concrete Institute (ACI) as a way of providing additional durability for concrete bridge decks.(1) At that time most experience with the technique had been in various applications of overlay concretes for the repair or major rehabilitation of old decks. In order to eliminate some variables such as penetrated chlorides and corroded steel that could affect the performance of overlays on older structures, the present study was initiated in 1974 and broadened in 1975 to evaluate the construction, concrete properties, condition, and performance of six two-course spans.(2,3)

The major benefits from two-course bonded construction are:

1. Placement of a protective overlay concrete above the reinforcing steel where needed to minimize the penetration of corrosive substances while allowing less costly ordinary concrete to be used for the bulk of the deck.

2. Isolation of common construction defects, such as cracking due to subsidence and plastic shrinkage, in the base layer concrete.

3. Provision of an option to use less costly aggregates, manufactured sands that polish and fail to maintain adequate skid resistance, or natural sands with high void percentages that require more mixing water and result in less durable concrete in the base layer, where they are acceptable.

4. Improvement in control of the depth of cover for the topmost reinforcing steel.

PURPOSE AND SCOPE

This report summarizes background findings already reported from the study concerning the evaluation of the construction, concrete properties, condition, and performance of the eight study spans through 1977. Also, comments are made concerning both other two-course construction projects completed in Virginia since the initiation of this study and the results of other studies as they pertain to bond strength requirements for overlays and to concrete cover depths for bridge deck reinforcing steel. Finally, an evaluation of the condition and performance of the study spans through 1980 is presented along with the conclusions and recommendations drawn from the study.

BACKGROUND

The following sections summarize information from published reports in this study as well as pertinent information from other research reports.

Project Description

The project plan for the six two-course spans is depicted in Figure 1, along with that for the two control, or ordinary, spans on nearby structures that were constructed within the same contract by a conventional, single-lift technique. Each of the three special portland cement concretes was applied as an overlay, or wearing course, for two of the two-course spans. The locations of the control and experimental spans were selected so that one of each type would be in both the EBL (eastbound lane) and WBL (westbound lane) to provide accountability for the effect of any differences in average service conditions between the lanes.

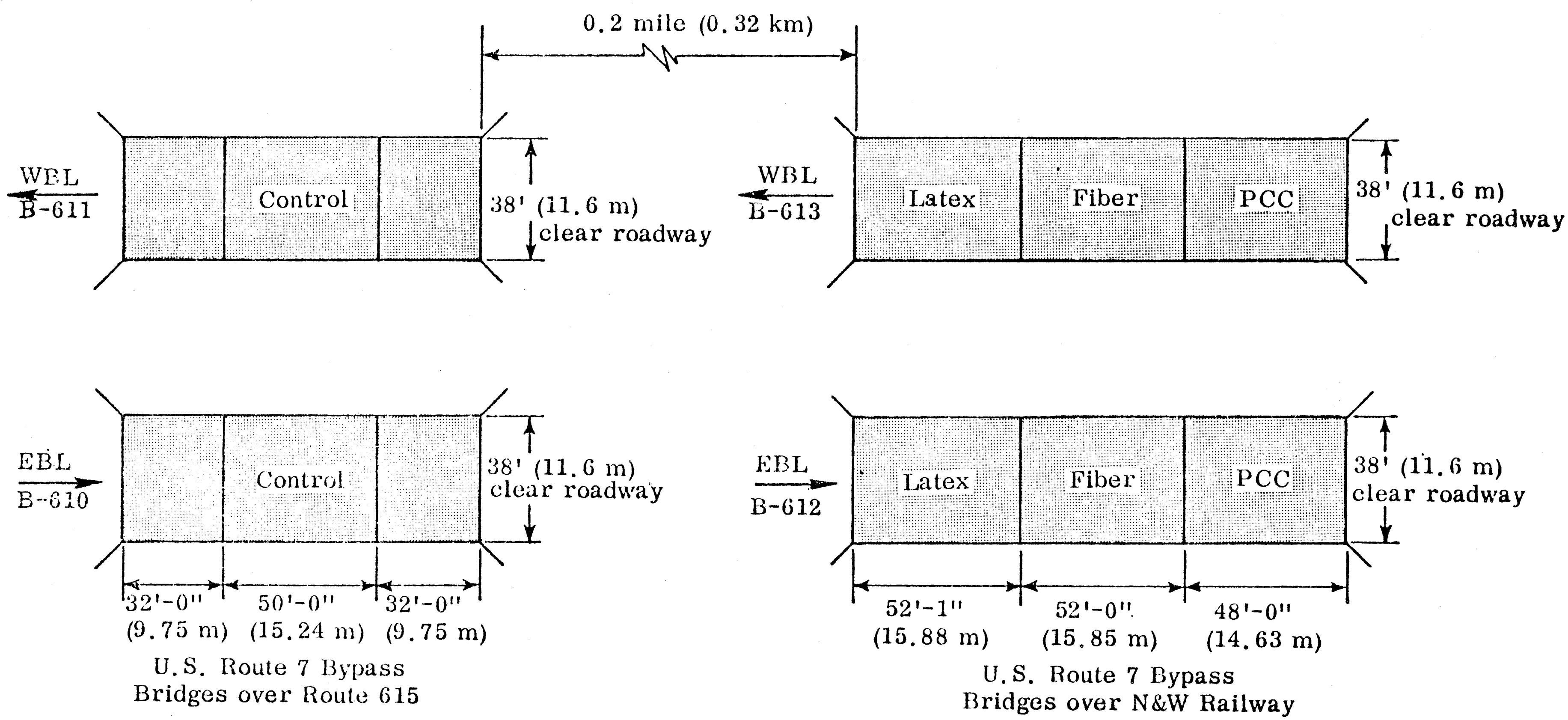

Figure 1. Plan view of bridge structures.

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The eight spans evaluated had a minimum design thickness of 8.5 in. (216 mm) and rested on simply supported steel girders having a center-to-center spacing of 8.5 ft. (2.5 m). The spans were approximately 50 ft. (15.2 m) in length with a clear roadway width of 38.0 ft. (11.6 m).

The overlay concretes evaluated are a latex-modified concrete, a low-water/cement (w/c) portland cement concrete (PCC), and a wire-fiber reinforced concrete. The latex was a styrene butadiene emulsion containing 46% to 59% solids and it was mixed at a rate of 24.5 gal.yd.^{3} (121.3 l/m^3) of concrete. The low-w/c PCC had a w/c = 0.41, as compared to a w/c = 0.47 for ordinary bridge deck PCC in Virginia, and a cement content of 705 lb./yd.^{3} (418 kg/m^3). The wire fibers were steel with 1.0-in. (25-mm) lengths and 0.016-in. (0.41-mm) diameters. The fiber content of this overlay concrete was 160 lb./yd.^{3} (95 kg/m^3).

Evaluation of Construction

The initial report on this study presented the results of an evaluation of the two-course construction technique conducted by comparing it with the single-lift technique typically employed in Virginia.(4) In the research then reported, the construction activities were analyzed to determine the relative influence of the two techniques on the times required to batch, deliver, and place the conventional and overlay wearing surface concretes.

The information presented in that report led to the observations and conclusions presented below.

1. The base layer concrete on the two-course spans was screeded satisfactorily using the transverse screed selected by the contractor for the overlay installations.

2. Light sandblasting of the base layers removed laitance that may have adversely affected the bond between the base layers and overlays.

3. The two-day delay between placement of a base layer and overlay is a desirable period. To avoid excessive drying, the bonding layer of cement slurry should be broomed onto the base layer not more than 10 ft. (3 m) ahead of nor longer than 15 minutes prior to the placement of overlay concretes.

4. The overlay concretes of the two-course spans were consolidated and finished satisfactorily using a transverse rotating drum screed with an attached surface vibrator unit.

5. The design strength of 4,000 psi (27.6 M Pa) at 28 days required in the structural concrete spans was exceeded significantly by the overlay concretes, thereby suggesting improved potential for good durability of these air-entrained mixes.

6. The clear concrete cover over the topmost reinforcing steel on the two-course spans was determined to be 2.3 in. (58.4 mm), of which 0.3 in. (7.6 mm) was mortar resulting from the base layer construction. This depth of cover is within one standard deviation of the average depth recorded for conventional single-lift bridge deck construction in Virginia.

7. Time and sequence data indicated similar, orderly progressions of construction activities for the two-course spans and the conventional single-lift spans.

8. A 7-yd.^{3} (5.4-m^3) truckload of concrete was screeded over 3½ times as much surface area for an overlay as for conventional single-lift spans; however, in both the two-course and single-lift techniques, the average times per truckload between the initial deposit and the completion of the screeding activity on the wearing surface were approximately the same.

9. Not including texturing and curing activities, the average number of man-hours required to install concrete in both layers of the two-course spans was 33% greater than for conventional single-lift spans.

10. In number of project days required for construction, the two-course and single-lift techniques are equivalent.

11. Coefficients of variation for the time intervals required to install each of the overlays and base layers were comparable to values representing excellent control for single-lift construction.

12. The three overlay concretes — latex-modified concrete, low-w/c PCC, and wire fiber-reinforced concrete — had placement characteristics compatible with the equipment and personnel employed in the two-course construction technique.

13. The batching activity was the only installation activity whose duration was significantly affected by differences in the three types of overlay concretes.

14. The total additional cost of the two-course technique was approximately 5% of the cost of the bridge deck superstructure.

The information in the initial report on the project demonstrated that from an operational standpoint, the two-course bonded construction technique is a reasonable alternative to conventional single-lift construction when additional protection of the upper reinforcing steel is warranted. Furthermore, because of the wide range of handling characteristics of the three overlay concretes used, it was suggested that any concrete having similar characteristics plus desirable properties as low permeability, good compressive and bond strengths, and freeze-thaw resistance could be successfully applied in the two-course technique.

Concrete Characteristics

The first objective of the second report on this project was to present information on the characteristics of the concretes in the two-course and control spans; the second objective was to present findings indicating the conditions of the decks prior to their being opened to traffic that would be useful in determining the performance of these spans.[5]

On the basis of information presented in that report, the following observations and conclusions were made with regard to the first objective, which was to present information on the characteristics of the concretes in the two-course and control spans.

1. The strengths, cement contents, and void systems of concrete cylinders cast from batches of the base layer and overlay concretes were analyzed and found to be within ranges indicative of good quality concretes. The relative quality of the conventional concrete was judged to be inferior, and some batches did not meet the stated specifications.

2. Prisms fabricated from batches of the overlay concretes were subjected to accelerated freezing and thawing in an NaCl solution, and the results indicated excellent potential for long-term durability of the field installations.

3. Depth of cover readings obtained on the hardened two-course decks, using the pachometer, were found to be equivalent to depths from direct probes made in the fresh concretes during construction, with the exception of the steel fiber-reinforced concrete,

which produced zero depth readings with the pachometer, and the depth of cover determinations by either technique may be compared directly to data for existing conventional decks.

4. Limited experimentation on this project with an acoustic monitoring device demonstrated the potential of such a device to assure the continuous and uniform functioning of vibratory equipment that is essential for producing adequately consolidated, well-bonded overlays. The presently available equipment would require modification and additional development.

5. The nuclear gage was used to check the degree of consolidation of the fresh overlay concretes and a reasonable range for specification control appears to be 100% ± 5% of the standard rodded unit weight of the fresh concretes.

6. Examinations of cores from the hardened two-course decks not only verified the adequacy of consolidation within the overlay concretes but also revealed the good condition of the bond at the interface between the overlays and the base layer concretes.

In general, it was concluded that the production of good quality overlay concretes was accomplished through the use of standard specifications supplemented by special contract provisions, and that the use of such specifications should be continued where two-course construction is warranted.

For decks as those in this study, it was suggested that a reasonable statement pertaining to the specification of cover over the topmost reinforcing steel would be that "The total cover over the topmost steel shall consist of not more than 0.3 in. (8 mm) of mortar cover resulting from the base layer construction and not less than 2.0 in. (51 mm) of the subsequently applied overlay concrete."

The importance of adequate consolidation of the overlays was emphasized and it was recommended that the nuclear gage currently available within the Virginia Department of Highways and Transportation should be used in a continuing evaluation to obtain information on which to base a specification for controlling the density of fresh concrete in two-course bridge deck overlays.

A council study was completed in 1980 in which the Troxler 3411 nuclear gage was evaluated for use in controlling the consolidation of PCC.(6) In general, the study showed that the precision of the nuclear gage was sufficient to provide density values within the expected range of variability obtained by other tests for density such as weight determinations for constant volumes. However, it was found that controlling density, per se, will not assure that adequate consolidation has been attained in all cases. The possible variability in the density of concrete resulting from acceptable variations in the grading and amounts of the individual ingredients, including air content, is of the same order of magnitude or greater than the variability in density resulting from poor consolidation.

Initial Condition of Spans

One year after construction, just prior to the opening of the roadway to traffic, several techniques were used to evaluate the condition of the six two-course spans and two single-lift control spans. Observations and conclusions from these activities are listed below as they appeared in fulfillment of the second objective of the second report in this study.(5)

1. Visual inspections and soundings of the two-course and single-lift spans indicated uniformly good quality concrete.

2. Chloride ions, attributable to the aggregates, were identified in the bridge deck concretes. Varying amounts of inert Cl^- are found in Virginia aggregates, and in the study spans the adjusted corrosion threshold wil be ... higher* than the Cl^- concentration at which active corrosion may normally be indicated.

*This statement was basically correct as it appeared in that report;(5) however, because of a change in the analysis procedure for Cl^-, the background Cl^- contents are slightly different from those reported then and are correctly stated later in this report for each of the study concretes.

3. The range of electrical potential readings on the two-course decks indicated a greater than 90% probability that no corrosion of the reinforcing steel was occurring, whereas on the single-lift control spans the range of readings indicated uncertainty about corrosion activity.

4. Sonic pulses transmitted through the spans resulted in ratings that indicated a better condition of the two-course spans relative to that of the conventional single-lift spans.

5. The skid resistance of the two-course spans and control spans was determined to be excellent.

The results of the initial condition survey of the study spans, along with the results of physical testing of the several concretes, were interpreted at that time to indicate a greater potential for satisfactory performance of the two-course spans than for the single-lift control spans.

It was recommended that the relative performances of the two-course and single-lift spans should be established by annual surveys of their condition.

Evaluation After Three Years

Some concern was expressed through the Council's Concrete Research Advisory Committee for the performance of two-course decks, particularly with regard to the bonding of the fresh to the hardened concrete. That concern resulted from the differing practices of several agencies for achieving bond. Among these practices were recommendations for roughening the surface of the base layer concrete in the fresh stage, sandblasting the hardened base layer, pre-wetting the base layer just prior to overlaying, and using various bonding agents such as cement slurry, grout, and epoxy.

This was the primary reason that the condition surveys were broadened in scope to include the measurement of sonic pulse velocities through the deck slabs. A significant decrease in pulse velocity would indicate, among other things, cracking and debonding between the base and overlay concrete. Therefore, the most important conclusion from the 1977 survey was that since the pulse velocities had shown no significant decrease, no change in the original good bond at the interface had taken place.

The results of other evaluations, as visual surveys and soundings, chloride contents, electrical potentials, and skid resistance, were also good, and it was stated that "based upon research and field experience to date, the concept of two-course construction for bridge decks is technically sound and should be considered where economically or technically justified."(7)

Other Bonded Deck Construction in Virginia

The use of bonded concretes for bridge deck construction has been evaluated in three situations in Virginia since the initiation of this study. Findings from the reports on those evaluations as they pertain to the bonding of fresh concrete to hardened concrete are discussed in the following sections.

Single-tee Bridge Superstructures

Precast, prestressed, concrete single-tee bridge superstructures have been constructed in Virginia in which the tee flange serves as the lower half of the bridge deck and the composite upper half is cast in place as a 4 in. (100 mm) thick reinforced concrete overlay. A report, issued in 1977, on this construction technique showed the results of flexural, shear, and tensile tests on laboratory models of the tee deck and described the condition of cores removed from a field structure. It was concluded that adequate composite action between the overlay and tee was achieved, even though concrete placed in two layers was found to be 20% to 60% weaker than concrete placed in one lift when the bond area was subjected to stresses that caused failure. It was recommended that the flanges of the tee beams be roughened when precast and that they be cleaned by sandblasting and pre-wetted with a water mist just prior to being overlaid. The use of a bonding agent, such as a cement slurry or grout, was not recommended because it did not increase bond strength.(8)

Internally-Sealed Concrete

Internally-sealed concrete was placed in 1977 as an experimental protective overlay for a three-span bridge deck using the same construction technique reported in this study, with the exception that no bonding agent was used. It was concluded in a report on that installation that the strength of the bond between the internally-sealed overlay concrete and the hardened base concrete was good, though no bonding agent was used nor was the base layer concrete roughened beyond the level resulting from normal screeding. In this case the base concretes were not sandblasted, although a water mist was sprayed to pre-wet the surface of the concrete. It was emphasized in the report that in any case if foreign materials or excessive laitance are present on the base layer surface, the deck should be sandblasted prior to placing the overlay concrete.[9]

Low-Slump Concrete

Low-slump concrete was installed in 1978 and 1979 as a protective system for a bridge deck. During the placement in the field, the low-slump concrete was investigated in the laboratory to determine a suitable procedure for bonding. Tests on field and laboratory specimens indicated that a slurry or grout was not needed to bond the low-slump overlay to the base concrete.[10]

During the first year, numerous cracks appeared in the low-slump overlays and hollow-sounding areas were detected. Issues the writer believes to be in need of attention are the macro-texture of the base concrete, the cleaning achieved by sandblasting, the amount of water sprayed on the base decks, the level of consolidation achieved in the base layer with the vibratory screed, and the levels of stress reached in various parts of that continuous deck as compared to stresses in the simple spans of this study. It is believed that failure of the overlays resulted from a combination of problems. The extremely rough texture of the base decks caused a weak bond area because the texturing weakened the base concrete and ponded water which produced additional weakening. Also, the structural design resulted in considerable stress in the bond area both during construction and after the deck was opened to traffic.

Bond Strength for Overlays

On the basis of his own work and a review of the reports of other researchers, the author has reached the general conclusion that the bonding of fresh concrete to hardened concrete can be readily accomplished either with or without a bonding agent, provided the base concrete is clean and sound and the overlay concrete is adequately consolidated.[11] This conclusion is consistent with findings from research by Felt.[12] For typical two-course deck construction, this has led the author to observe that bond shear strengths of 200 and 400 $lb.in.^2$ (1.38 and 2.75 MPa) should be interpreted as good and excellent, respectively, because the level of stress at the bond zone theoretically should be much lower.

A recent publication by the Transportation Research Board (TRB) deals directly with this issue and the following paragraph is quoted from that source except that references, where cited, are omitted.

> The reason that adequate bond strength can be achieved in a number of different ways is that, provided the deck is properly prepared, bond strengths are considerably in excess of the maximum shear stress at the bond line. The horizontal shear at the interface between a 7-in. (180-mm) thick uncracked slab and a 2-in. (50-mm) thick overlay has been estimated to be 64 psi (440 kPa) under an AASHTO H20 wheel load plus impact. Other work has indicated that a bond strength as low as 40 psi (280 kPa) may be adequate for an overlay. Shear bond strengths measured in the laboratory and using any of the previously mentioned bonding agents are typically in the range of 350 to 500 psi (2.4 to 3.4 MPa). This is not to suggest that bonding procedures can be compromised. It simply indicates that, provided the base concrete is clean and sound and good construction procedures are employed, the bond strength between the base course and a concrete overlay has a considerable safety factor.[13]

Depth of Concrete Cover

The depth of concrete cover over the topmost reinforcing steel for the eight study spans was specified indirectly as 1.94 in. (49 mm) on the basis of the design plans, which specified the total deck thickness and the placement locations for the reinforcement. For the six two-course spans it was specified additionally that the concrete in the first lift would be struck off level with the topmost reinforcement and would be overlaid with one of the three types of concrete.

The average cover depths and standard deviations that resulted from this specification were shown in an earlier report and are given here in Figure 2. In that report, a better statement pertaining to the specification of covers over the topmost reinforcing steel would be that "The total cover over the topmost steel shall consist of not more than 0.3 in. (8 mm) of mortar cover resulting from the base layer construction and not less than 2.0 in. (51 mm) of the subsequently applied overlay concrete."

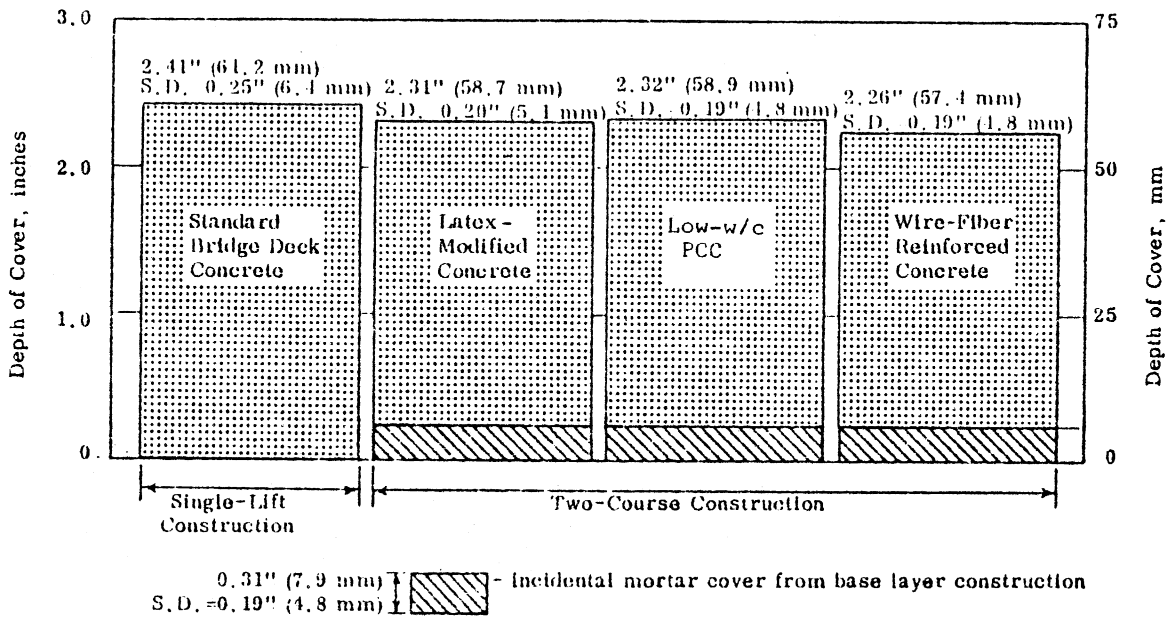

Figure 2. Average depth and variability of cover over topmost reinforcing steel for eight study spans.

A required minimum depth of 2.0 in. (51 mm) for overlay concretes would necessitate a target, or specified, overlay depth of approximately 2.9 in. (74 mm) based on the standard deviation of 0.19 in. (4.8 mm) shown in Figure 2. If the value of the standard deviation for the depth of overlay concrete is greater, as extensive studies of cover depth for reinforcement in ordinary single-lift decks suggests it may be, then the target depth would obviously be greater than 2.9 in. (74 mm). Whatever the correct value of the standard deviation, either for two-course or single-lift construction, it must be taken into account if a rational decision regarding the specified depth of cover is to be made.

In the following sections summaries of other research are presented which indicate probable values for the standard deviation of cover depths associated with ordinary single-lift construction and which suggest the minimum cover depths to be specified for bridge deck reinforcement.

Variability of Cover Depth

In the early 1970's, a survey of concrete bridge decks in Virginia revealed that for decks with a specified cover of 1.94 in. (49 mm), the average depth of cover was 2.50 in. (64 mm) with a standard deviation of 0.48 in. (12 mm). A cumulative frequency plot of the normally distributed cover depths indicated that of the deck areas represented, only 16% had less than the specified cover.[14]

During the same period a survey was made of 17 bridge decks in New Jersey. It was found that for individual bridge decks, the standard deviations for depth of cover ranged from 0.18 to 0.48 in. (4.6 to 12.2 mm) and the means ranged, both plus and minus, up to 0.38 in. (9.5 mm) from the specified value. An operating characteristic curve was developed from the survey data to indicate the required, or specified, depth of cover to assure a minimum 2.0-in. (51-mm) depth of cover for any selected percentage of the topmost steel. For example, with a specified depth of 2.5 in. (64 mm), this minimum cover could be assured for 90% of the steel.[15]

The ACI has recommended that a minimum cover of 2.0 in. (51 mm) be provided and has stated that "a variation of ± 0.5 in. (13 mm) might be considered normal."[1]

Required Cover Depth

A synthesis on the durability of concrete bridge decks published in 1979 by the TRB recommends that research be directed to construction practices, specifically to improved methods of fixing the steel and placing the concrete, to ensure that the design cover is achieved.[13]

That publication also indicates a need to assess the role of concrete quality and cover in determining whether spalling necessarily occurs if the reinforcing steel corrodes. The point made is that active corrosion potentials are known to have existed for several years in numerous bridge decks which have in excess of 2.0 in. (51 mm) of good quality concrete cover, yet no physical distress has occurred.

With concretes having various qualities, the time from construction until active corrosion potentials of the reinforcing steel are recorded has been extensively studied by Spellman and Stratful.[16] It appears from that work that active potentials can be expected within two years for structures with normal cover and even for a w/c as low as 0.35. A standard procedure for measuring, interpreting, and reporting electrical potentials of reinforcing steel in concrete has been adopted by the ASTM.[17]

A comprehensive investigation was made by the Federal Highway Administration (FHWA) to obtain information on numerous approaches for protecting bridge deck reinforcement in corrosive environments. For test slabs subjected to 830 daily salt applications it was reported that 95% of the reinforcing steel could be protected from chloride-induced corrosion with cover consisting of ordinary concretes having w/c's of 0.4, 0.5, and 0.6 and corresponding minimum cover depths of 1.7, 2.8, and 3.1 in. (43, 71, and 79).[18]

For various reasons, the minimum necessary cover depths indicated by the FHWA report for specified w/c's are probably not conservative as discussed in that report, and a recently completed survey in Virginia of bridge decks in service for seventeen years supports the FHWA findings.[19] On the basis of average chloride ion contents, the results of the Virginia survey may be interpreted to show that for these decks a minimum cover of 2.4 in. (61 mm) would have allowed most of the reinforcement to now be below a depth at which the corrosion threshold, 1.3 lb.Cl^-/yd.^{3} (0.77 kg Cl^-/m^3) of concrete, would be reached due to the penetration of chlorides.

Cost of Additional Cover

In 1976, the cost of increasing the depth of concrete cover for bridge decks in Kansas was evaluated on the basis of minor design modifications for conventional continuous spans.[20] It is the author's understanding that this alternate design was not implemented because it did not satisfy federal guidelines for bridge deck protective systems.[21]

The modification involved lowering the upper reinforcing steel 1.0 in. (25 mm) so that the cover specified could be increased from 2.0 in. (51 mm) to 3.0 in. (76 mm) while maintaining the original deck thickness. The negative moment steel was increased by one bar size in this alternate design. Additionally, the w/c of the concrete was specified to be 0.35 instead of 0.44. The normal slump of 2 to 4 in. (50 to 100 mm) was to be maintained by increasing the cement factor approximately 30%. The projected service life for the alternate design was three times that of conventional decks with an estimated 2% increase in construction cost.[20]

The additional dead load associated with a 1.0 in. (51 mm) increase in deck thickness to achieve a 1.0 in. (51 mm) increase in cover would be a uniformly distributed 500 lb./ft. (744 kg/m) of span length for typical two-lane structures. This could result in a minimum cover, based on a standard deviation of 0.48 in. (12.2 mm), of approximately 2.4 in. (61 mm) for 95% of the topmost reinforcing steel in bridge decks in Virginia as compared to the slightly more than 50% receiving that minimum cover in earlier years.[14]

The plans for bridge decks in Virginia were modified by 1976 to indicate the dimension from the center of the topmost reinforcing bar, usually a No. 5 (16-mm)diameter bar, to the finished deck surface as 2.75 in. (70 mm). Therefore, the specified cover is now 2.44 in. (62.0 mm).

Contractors who construct bridge decks in Virginia are awarded a bonus for up to 0.5 in. (13 mm) of additional deck thickness as compared to the thickness shown in the plans. This provides an incentive to avoid shallow cover. It is believed that this approach has improved cover depths in Virginia; however, a comprehensive survey will have to be conducted to determine the average and the variability of the depth of cover being provided.

CONDITION AND PERFORMANCE

The condition of the control and two-course spans was determined in 1980 on the basis of tests as skid resistance, soundings, visual surveys, sonic pulse transmissions, water absorption, chloride content, and electrical potentials. The information obtained was compared to similar data from previous years to evaluate the performance of the study spans. The test data are presented and discussed in the following sections.

Skid Resistance

The skid resistance of each study span was determined in accordance with ASTM E274 and the resultant skid numbers are shown in Table 1. Each number is an average of five tests and, as indicated in the table, both the traffic (T) and passing (P) lanes of the eastbound and westbound deck surfaces were tested. For the treaded tire all skid numbers in 1975 were approximately equal to 60, which is indicative of excellent skid resistance. In 1977 there were slight reductions in skid number values, particularly in the traffic lane, as expected. The 1980 tests showed ranges of skid numbers of 48 to 52 and 59 to 63 in the traffic and passing lanes, respectively. Both ranges are indicative of excellent skid resistance. No significant differences were indicated among the various types of concretes used in the overlays and control spans.

Skid tests with smooth tires were also conducted in 1980. These numbers also indicate excellent skid resistance for all spans, with the highest skid numbers being found on the control spans and the lowest on the wire-fiber reinforced concrete surfaces.

Table 1

Results of Tests on Study Spans in 1975, 1977, and 1980 with Skid Trailer at 40 mph (64 km/hr.)

Span	Lane	Average Skid Trailer Numbers*			
		Treaded Tire			Smooth Tire
		1975	1977	1980	1980
Single-Lift:					
Control	EBTL	56	53	50	31
	EBPL	59	55	63	43
	WBTL	56	46	49	27
	WBPL	58	59	61	38
Two-Course:					
Latex-Modified	EBTL	62	55	50	30
	EBPL	64	59	59	34
	WBTL	60	51	52	25
	WBPL	61	57	61	33
Low-w/c PCC	EBTL	60	52	51	25
	EBPL	62	54	61	29
	WBTL	62	53	51	29
	WBPL	62	63	59	35
Wire-Fiber	EBTL	61	52	48	21
	EBPL	63	56	60	29
	WBTL	61	52	52	24
	WBPL	61	60	59	30

*1975 and 1977 skid numbers from VHTRC trailer; 1980 skid numbers from VDHT trailer no. 2.

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Soundings

The deck surfaces of the control and two-course spans were sounded using a chain drag, a hammer, and a rolling delamination detector(22) loaned by the FHWA. Three small unsound areas were located, one on each of three spans representing each of the overlay materials, as indicated in Figure 3.

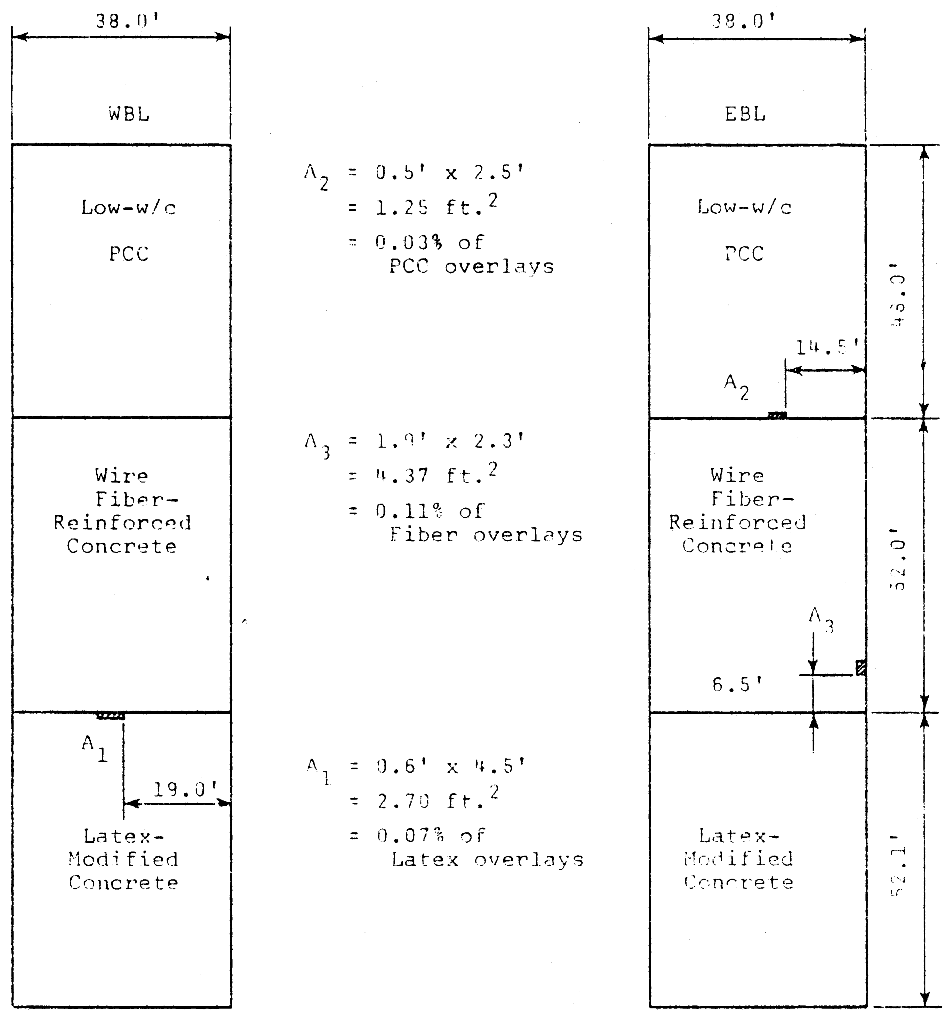

Figure 3. Unsound areas on two-course study spans detected using chain drag, hammer, and FHWA "Delamtect".
Note: ft. x 0.3 = m
$ft.^2 \times 0.09 = m^2$

One of these small areas, A_3 in Figure 3, had been located in 1977 and coring revealed that the hollow sound was due to honeycombing above the bond zone because of inadequate consolidation. It is probable that areas A_1 and A_2 also are unsound because of honeycombing that has existed from the time of construction.

At this time, six years after construction, it is believed that these small unsound areas — representing 0.07%, 0.03% and 0.11% of the latex, PCC, and fiber overlays, respectively — are insignificant both in terms of the consolidation during construction, which was reported earlier as satisfactory, and with respect to performance, which the soundings revealed to be good for more than 99% of the areas on the three types of overlays. The control spans had no unsound areas.

Visual Inspection

A visual inspection of the study spans revealed no significant defects other than cracking, as summarized in Table 2. Individual cracks ranged primarily from 1.0 to 6.0 ft. (0.3 to 1.8 m) in length and were generally oriented in either the transverse and longitudinal directions with respect to the bridge deck centerline, and were randomly distributed over the surface. An exception to these observations was on the EBL control span,where pattern cracks were found over the majority of the span surface but not measured, and on which 84% of the measured transverse cracks were located in the traffic lane.

The surface cracks on the study spans are attributed to plastic shrinkage of the concretes. The total plastic shrinkage that each of these concretes could experience was dependent on its mixture proportions, and on this basis it is reasonable to state that no great differences existed among these concretes in the fresh stage. Moreover, the major factor, loss of water through evaporation from the concrete surface, affecting the plastic shrinkage of these concretes was well within the control of good curing practices, as is discussed in the following paragraphs.

The method of ensuring a satisfactory cure for concretes in the study spans differed for the control and overlay concretes. The control spans were cured with the application of a sprayed white liquid membrane seal and the overlay concretes with moist burlap covered with white polyethylene sheeting. The moist burlap was maintained for 7 days, except for that on the latex-modified concrete, which was removed after 24 hours in accordance with the instructions of the latex manufacturer.

Table 2

Surface Cracks Observed

Span	Lane	Total Crack Lengths, ft.	
		Transverse	Longitudinal
Single-Lift:			
Control	EBL*	75.1**	None
	WBL	None	None
Two-Course:			
Latex-Modified	EBL	5.3	5.0
	WBL	17.5	12.8
Low-w/c PCC	EBL	6.7	1.0
	WBL	None	2.0
Wire-Fiber	EBL	None	None
	WBL	None	None

*Pattern cracks observed over majority of span surface but not measured.

**84% located in traffic lane.

NOTE: ft. x 0.305 = m

The crack lengths shown in Table 2 suggest that reasonable control of curing was achieved for all spans other than the EBL control span and the WBL latex-modified concrete overlay, and comments on these two spans are made in the next two sections. Each curing method is believed to be capable of providing a satisfactory cure,and the sprayed liquid membrane seal would probably be satisfactory on all of the overlaid spans except that with the latex-modified concrete. Both curing methods are included, among others, in the ACI's most recent proposed standard for curing concrete.[23]

Cracking on EBL Control Span

The poor performance, in terms of cracking, of the EBL control span can be explained with the aid of records for construction activities, concrete properties, and weather conditions maintained at the time of placement.[4]

The times to batch, deliver, deposit, screed, texture, and apply curing materials were recorded and the times for these activities, from the start of batching through the completion of screeding, are shown in Figure 4a as an average for all batches of each type of concrete (control, base, latex, PCC, and fiber). The largest differences in time result from the fact that the fibrous concrete required special batching procedures and the latex-modified concrete, which was batched from mobile continuous mixers on the site, had no delivery time associated with its placement. Also, the average time shown in Figure 4a for the latex-modified concrete was computed for approximately eight discrete batches from each of two truckloads instead of for entire truckloads. Therefore, in order to compare the several concrete placements in terms of the average times the concretes were being manipulated on the spans, and consequently the average times each was exposed to drying conditions, the average times in Figure 4b have been computed from the time of initial deposit to the completion of screeding for individual truckloads of concrete. The average times for this important phase of handling the concretes are seen in Figure 4b to be approximately equal.

The evaporation rates of water from the fresh concretes for each placement situation, which include different days for the two controls and different periods of the same day for each pair of overlay placements, are shown in Table 3. The rate of evaporation in each case was determined graphically using recorded values of air temperature, relative humidity, concrete temperature, and wind velocity, and a nomograph published by the ACI.[24] The ACI suggests that if the rate of evaporation approaches 0.2 lb./ft.^{2}/hr. (1.0 kg/m^2/hr.), precautions should be taken to avoid plastic shrinkage cracking, because this rate of evaporation will likely exceed the rate at which free water can bleed from the fresh concrete.

> These precautions consist of dampening subgrade and forms, placing concrete at the lowest practicable temperature, erecting windbreaks and sunshades, reducing time between placement of concrete and start of curing, and minimizing evaporation, particularly during the first few hours subsequent to placing concrete, by a suitable means such as applying moisture by fog spraying.[24]

The rates of evaporation, as shown in Table 3, were moderate for the two control spans, severe for the EBL wire-fiber span, and low to moderate for the other four spans.* The absence of cracking, shown in Table 2, on the EBL wire-fiber span is attributed to

*Sufficient data were not recorded to allow a determination of the rate of evaporation for the EBL latex-modified concrete.

the short delay between placement and the application of curing materials, although the wire fibers may have helped to inhibit cracking.(4)

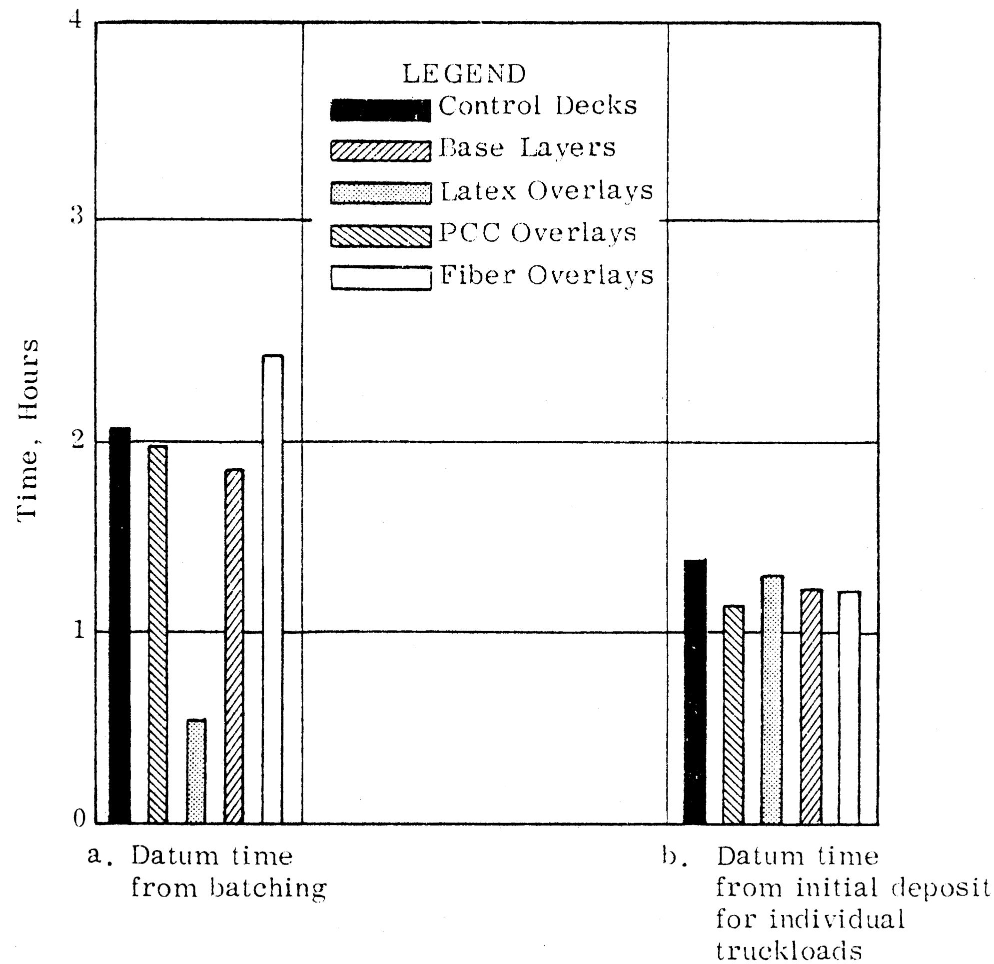

Figure 4. Average times to completion of the screeding activity.

*A batch and truckload of concrete are the same, except in the case of latex-modified concrete where, using mobile continuous mixer trucks in lieu of ready-mix plant trucks, approximately eight discrete "batches" were discharged from each truckload.

Table 3

Rate of Evaporation of Water From Fresh Concretes

Span	Lanes	Rate of Evaporation, lb./ft.^{2}/hr.*
Single-Lift:		
Control	EBL	0.12
	WBL	0.12
Two-Course:		
Latex-Modified	EBL	—
	WBL	0.08
Low-w/c PCC	EBL	0.04
	WBL	0.06
Wire-Fiber	EBL	0.18
	WBL	0.04

*lb./ft.^{2}/hr. x 4.88 = kg/m^2/hr.

While Table 3 shows that the two control spans were subject to the same rate of evaporation, it will be recalled from Table 2 that the EBL span experienced serious cracking and the WBL span had none. The complete record, shown in Figure 5, for the sequence and times for installation activities for the control spans helps to explain the cracking. In Figure 5 it may be observed that the average delay between completion of the screeding activity and application of the curing material was 2.2 hours for the EBL span versus 1.3 hours for the WBL span. This additional delay, the reasons for which are unclear, would have increased the likelihood of plastic cracking of the EBL span. Also, the average compressive cylinder strengths for the EBL span were significantly better, 500 psi (3.4 MPa), than those for the WBL span, which did not crack. The higher compressive strength suggests better control of mixing water for the EBL control span and, ironically, a greater tendency for plastic cracking because of its lower free water content.

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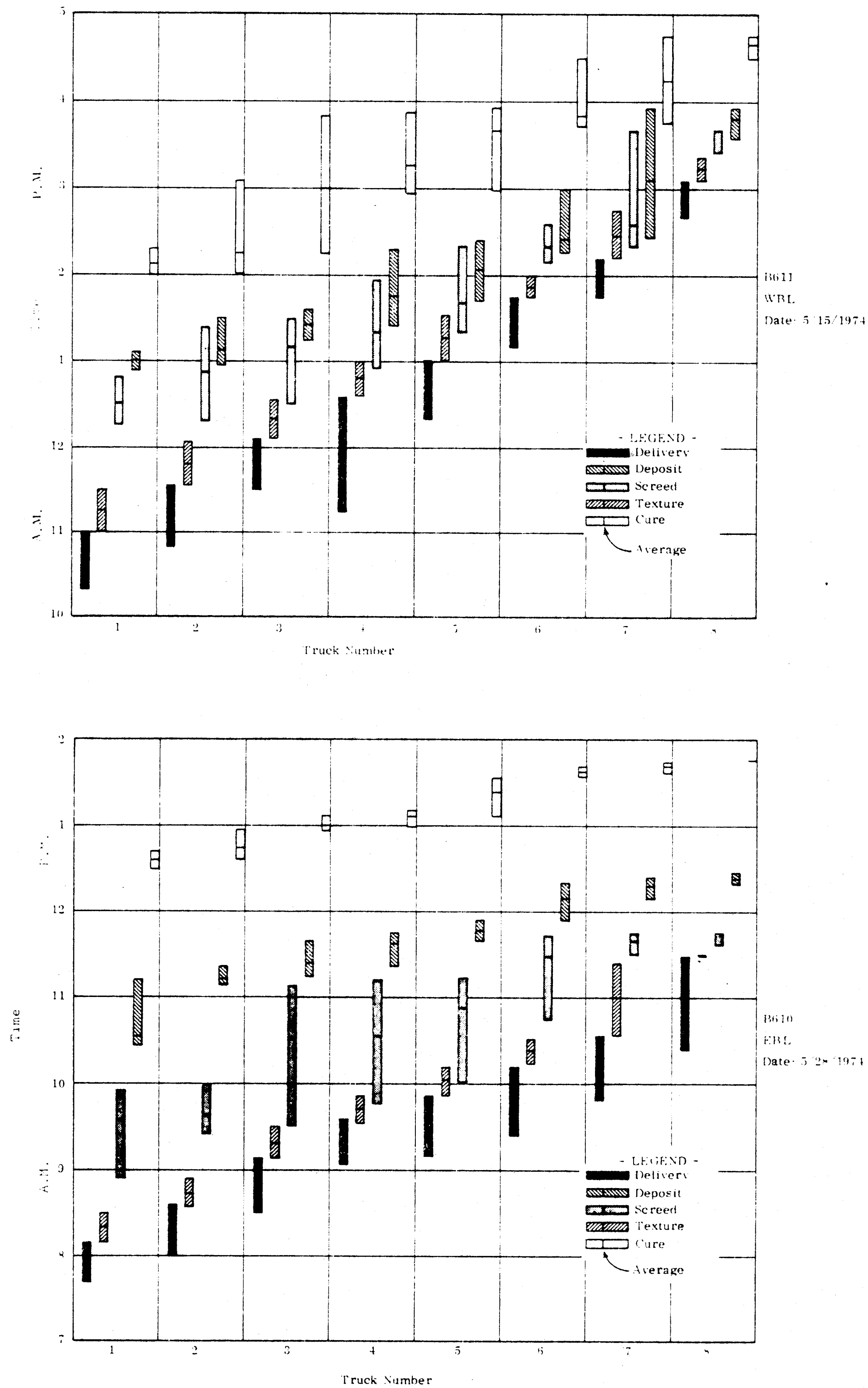

Figure 5. Sequence and times for installation activities for control spans (WBL span above, EBL span below).

The cracking of the EBL control span serves to emphasize the need for control in the application of curing materials. It is believed that both the moist-curing and the membrane-curing methods will give satisfactory results if properly applied for both single-lift and two-course construction. There is no basis in this study for suggesting that the tendency for plastic cracking is affected by two-course construction; however, the relationships between subsidence cracking of single-lift placements and several variables have been documented.[25]

Cracking on WBL Latex-Modified Span

The amount of cracking on the WBL latex-modified concrete span was much less than that on the EBL control span; however, it was significantly greater than the amount on the other overlaid spans, as is shown in Table 2. The reason for this is not clearly discernible from the construction records. Installations of latex-modified concrete should be monitored during and after construction to determine the particular conditions that may predispose them to cracking.

Sonic Pulse Velocities

The travel times for sonic pulses transmitted vertically from the bottom to the top of the 8.5 in. (216 mm) deck thickness were recorded at 44 locations on each study span. The pulse travel time in concrete is increased as the effective path is lengthened by voids and cracks in the matrix. The pulse velocity is calculated using the nominal path length between the transmitter and receiver. The velocities may be used to evaluate the uniformity of the concrete in a structure and to rate the general quality of the concrete on the basis of the sonic pulse velocity ranges given in Table 4.[26] A standard test method for determining the velocity of the propagation of compressional waves in concrete is given by ASTM C597-71.

The sonic tests were made not only to assess the uniformity and relative quality of the concretes in each span, but also to provide a means of determining changes in the pulse velocities with time. A significant decrease in pulse velocity could indicate cracking from any cause and, for the two-course spans, deterioration of the bond between the overlay and base deck.

The percentage of sonic pulse velocities falling within the several quality ranges in 1975 are listed in Table 5 for each of the study spans. These initial velocities were determined approximately one year after construction so that changes observed in subsequent readings due to maturity of the concretes would be minimized. It is apparent that the average quality of the two-course spans was better than that of the single-lift control spans and that the two-course concretes were of a more uniform quality.

Table 4

Ratings for Ranges of Sonic Pulse Velocities through Concrete (Reference 26)

Pulse Velocity, ft./sec.*	Quality
Above 15,000	Excellent
12,000 - 15,000	Good
10,000 - 12,000	Questionable
7,000 - 10,000	Poor
Below 7,000	Very Poor

*ft./sec. x 0.305 = m/sec.

Table 5

Sonic Pulse Ratings for Study Spans in 1975

Span	Lane	Percentage of Sonic Pulse Velocities in each Range				
		Excellent	Good	Questionable	Poor	Very Poor
Single-Lift:						
Control	EBL	—	20	52	26	2
	WBL	—	12	51	30	7
Two-Course:						
Latex-Modified	EBL	2	83	15	—	—
	WBL	—	59	41	—	—
Low-w/c PCC	EBL	—	77	23	—	—
	WBL	—	50	50	—	—
Wire-Fiber	EBL	—	100	—	—	—
	WBL	15	34	34	17	—

In Table 6 the percentages of sonic pulse velocities in each range are listed for the spans from tests performed in 1975, 1977, and 1980. A statistical analysis would be required to determine the mathematical significance of the shifts in percentages; however, a qualitative assessment is that a probable decrease in quality is indicated for the WBL latex-modified and WBL wire-fiber spans, and that these decreases are equivalent to the decreases seen for the single-lift control spans and are, therefore, not uniquely associated with the two-course bonded construction.

Absorption

The spans were cored one year after construction and the cores were used to determine the void contents of the several hardened concretes and the condition of the bond interface for the overlay concretes. The void contents were found to be satisfactory and the bond zones were of excellent quality as reported earlier.[5]

An additional 4.0-in. (100-mm) diameter core was drilled from each study span in 1980 and tested to determine the water absorption values for the several concretes. A standard procedure for determining the absorption of hardened concrete is given in ASTM C642-75, although different procedures for obtaining and handling specimens, plus the distinction between absorption and permeability, lead to a general statement that absorption alone cannot be used as a measure of the quality of concrete.[27] The discussion of concrete quality in terms of its absorption is justified, however, on the basis of recent findings by Newlon from a study of the performance of normal bridge deck concretes exposed for 17 years to typical service environments.[19] Those findings suggest that bridge deck reinforcement having a 2.4-in. (61-mm) minimum cover depth of concrete with an absorption not exceeding 4.5% to 5.0% is adequately protected for periods approaching 20 years from penetrated chloride ion concentrations considered sufficient for inducing corrosion.

The values for absorption at the 2.0- to 4.0-in. (50- to 100-mm) level in each of the study spans are shown in Table 7 to be approximately equal to 6.0%, although the absorptions for the control spans are slightly higher than 6.0% and all of the two-course samples, representing the base layer concretes for this depth, have absorptions less than 6.0%. Many factors, such as consolidation and curing, may influence absorption, but it is believed that the project records clearly support the conclusion that the w/c's of the single-lift control spans were higher than those for the base layers of the two-course spans.[5] An underlying cause of this situation was the higher water demand of the non-polishing siliceous fine aggregate used in the control spans versus that of the polishing calcareous fine aggregate allowed in the base layers, where skid resistance is not a consideration.

Table 6

Sonic Pulse Ratings for Study Spans in 1975, 1977, and 1980

Span	Lane	Percentage of Sonic Pulse Velocities in Each Range for Each Year														
		Excellent			Good			Questionable			Poor			Very Poor		
		75	77	80	75	77	80	75	77	80	75	77	80	75	77	80
Single-Lift:																
Control	EBL	--	--	--	20	--	--	52	86	58	26	14	42	2	--	--
	WBL	--	--	--	12	13	--	51	79	17	30	8	70	7	--	13
Two-Course:																
Latex-Modified	EBL	2	--	--	83	36	100	15	55	--	--	9	--	--	--	--
	WBL	--	--	--	59	29	17	41	58	58	--	13	25	--	--	--
Low-w/c PCC	EBL	--	--	--	77	71	100	23	29	--	--	--	--	--	--	--
	WBL	--	7	--	50	67	100	50	26	--	--	--	--	--	--	--
Wire-Fiber	EBL	--	--	--	100	77	96	--	23	4	--	--	--	--	--	--
	WBL	15	--	--	34	21	23	34	46	23	17	33	46	--	--	8

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Table 7

Absorption Values from Cores Obtained in 1980

Span	Lane	Absorption, % of Dry Weight — Sample Depth: 0.0 to 2.0 in. Individual	0.0 to 2.0 in. Avg.	2.0 to 4.0 in. Individual	2.0 to 4.0 in. Avg.
Single-Lift:					
Control	EBL	5.7	6.3	6.1	6.4
	WBL	6.6		6.6	
Two-Course:					
Latex-Modified	EBL	1.2	1.1	5.9	5.6
	WBL	1.0		5.2	
Low-w/c PCC	EBL	4.8	5.2	5.8	5.8
	WBL	5.6		5.8	
Wire-Fiber	EBL	6.2	6.3	5.6	5.6
	WBL	6.3		5.6	

NOTE: in. x 25.4 = mm

The non-polishing siliceous fine aggregate used for the overlays of the two-course spans was required to have a better particle shape, and consequently, lower water demand, than that of fine aggregate normally used in bridge deck construction in the area including the study site. This and an 11% increase in cement content allowed the low-w/c (0.41) PCC to be placed with a slump range of 2.7 to 3.3 in. (69 to 84 mm) and resulted in improved quality by comparison to the control spans, as indicated by the average absorption value of 5.2% as shown in Table 7.

The average absorption value of 6.3% for the wire fiber-reinforced concrete overlays was equal to that for the control spans; however, the average absorption of 1.1% for the latex-modified concrete was much lower than any other absorption value.

It should be emphasized that absorption values of 6.3% are representative of the most absorptive bridge deck concretes in Virginia; however conditions were such that this value was also representative of ordinary construction in the area of the study site.

The observations and conclusions made earlier in this section should be understood from that perspective. Also, even though the latex-modified concrete had extremely low absorption values, as shown in Table 2, similar to those to be expected for polymer-impregnated concrete and internally-sealed concrete,[30,9] it will be shown later in this report that the chloride ion penetration into this concrete, albeit generally lower, is not always lower at depths in the concrete approaching the level of the reinforcing steel.

The absorption test provides a convenient indirect means for indicating the effectiveness of the latex modifier in a concrete known to contain it. The latex content of a concrete may be determined by chemical analysis; however, this involves the use of equipment not available at the Council, and the analysis alone would not show conclusively that the latex, even if uniformly distributed, was functioning in the intended manner. Other researchers have demonstrated the usefulness of an electron microscope for verifying the bridging action of polymer microfibers across microcracks in a latex-modified concrete.[28] This technique was attempted in this study; however, no microfibers were visible in micrographs with a 4800X magnification of a fractured surface of latex-modified concrete from a depth of 0.5 in. (13 mm) in the EBL study span.[29]

The continued use of polymer-modified concretes, either in Virginia or elsewhere, would suggest the need for convenient and reliable test procedures for quality assurance. In Virginia the accessibility of an electron microscope would allow the convenient use of micrographs, with no investment in equipment, as part of a quality assurance program. Additional research will be required to determine if such a program is feasible.

In addition to possibly providing a means for assuring the quality of polymer-modified concretes, micrographs of the paste structure of these and other concretes might yield information relevant to the role of absorption as an indicator of the potential performance of the concretes.

Chloride Contents

The chloride ion contents of the experimental and control concretes were determined for samples drilled from the field structures and from corresponding reinforced deck models fabricated at the time of construction and exposed to deicing salt applications throughout the ensuing six-year period in an outdoor storage area at the Research Council. Sampling depths of 0.5 to 1.0 in. and 1.5 to 2.0 in. (13 to 25 mm and 38 to 51 mm) were used, and the penetrated Cl^- content determinations are shown in Table 8 as an average for four samples (two each from both the EBL and WBL spans and corresponding deck models). These results were computed by subtracting the background Cl^- contents shown in Table 8 from the total Cl^- contents found in the samples.

Table 8

Average Cl^- Contents of Study Concretes in 1980

Span	Chloride Content, lb. $Cl^-/yd.^3$		
	Background	Penetrated	
		0.5 to 1.0 in.	1.5 to 2.0 in.
Field Structures:			
Single-Lift:			
Control	0.4	0.3	0.2
Two-Course:			
Latex-Modified	0.3	0.1	0.2
Low-w/c PCC	0.4	0.6	0.4
Wire-Fiber	0.5	1.4	0.1
Deck Models:			
Single-Lift:			
Control	0.4	2.4	0.1
Two-Course:			
Latex-Modified	0.3	0.0	0.2
Low-w/c PCC	0.4	0.4	0.1*
Wire-Fiber	0.5	2.2	1.8

*Based on average of three samples instead of four.

NOTE: in. x 25.4 = mm

lb. $Cl^-/yd.^3$ x 0.59 = kg Cl^-/m^3

The results in Table 8 for the field structures indicate small amounts of Cl^- at the greater sampling depth for each concrete by comparison to a corrosion threhold value of 1.3 lb.Cl^-/yd.^{3} (0.77 kg Cl^-/m^3) of concrete. This finding is verified by the Cl^- concentrations found in the deck models at the greater sampling depth, an exception being for the wire-fiber reinforced deck models, where inadequate consolidation may have been the factor that resulted in 1.8 lb. Cl^-/yd.^{3} (1.1 kg Cl^-/m^3) of concrete penetrating to the greater sampling depth.

The shallower sampling depth was in some cases found to contain high concentrations of Cl^-, which are not necessarily indicative of the relatively low concentrations found at the greater depth. This finding can be attributed to variations in factors such as the w/c and consolidation within each concrete, which would reduce their ideal resistance to Cl^- penetration.

The chloride data for the study concretes can be compared with the data from exhaustive investigations of Cl^- penetration into hardened concrete performed by Clear,[18] and with Cl^- penetrations observed during a long-term evaluation of bridge deck performance in Virginia by Newlon.[19] In Figure 6a the Cl^- concentrations in the field structures in 1980 have been plotted as a function of average sampling depth. Each concrete exhibits a plot characteristic of a low rate of Cl^- penetration, with maximum Cl^- contents near the level of the reinforcing steel being less than 0.5 lb. Cl^-/yd.^{3} (0.3 kg Cl^-/m^3) of concrete, an exception being the wire-fiber reinforced concrete, which although having a low chloride content near the level of the steel, exhibits a plot characteristic of rapid Cl^- penetration. The plots in Figure 6b, representing the harsher exposure of the deck models by ponding as compared to the normal exposure of the field structures, indicates a high resistance to Cl^- penetration by the latex-modified concrete and low-w/c PCC; however, the wire-fiber reinforced concrete has a chloride content approaching the chloride corrosion threshold near the level of the reinforcing steel, and the ordinary PCC exhibits a plot characteristic of a rapid rate of Cl^- penetration.

Even though the wire fiber-reinforced concrete has high Cl^- contents, the steel fibers have corroded only where exposed at the deck surface. This observation confirms laboratory findings, reported earlier in this study, which showed that wire fiber-reinforced concrete specimens did not exhibit corrosion of the steel fibers deeper than approximately one fiber diameter beneath the surface of the specimen.[4] The fibers seem to be protected from corrosion by the intimate fiber-paste contact, despite the high Cl^- contents in the paste.

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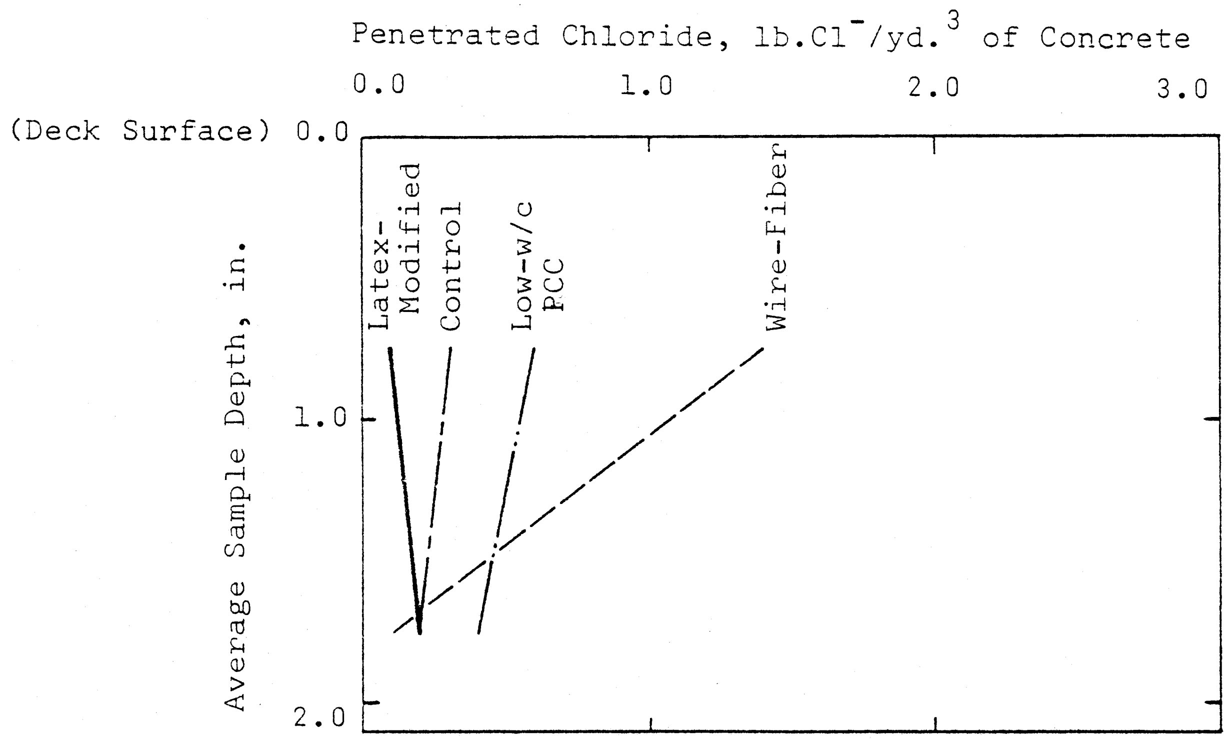
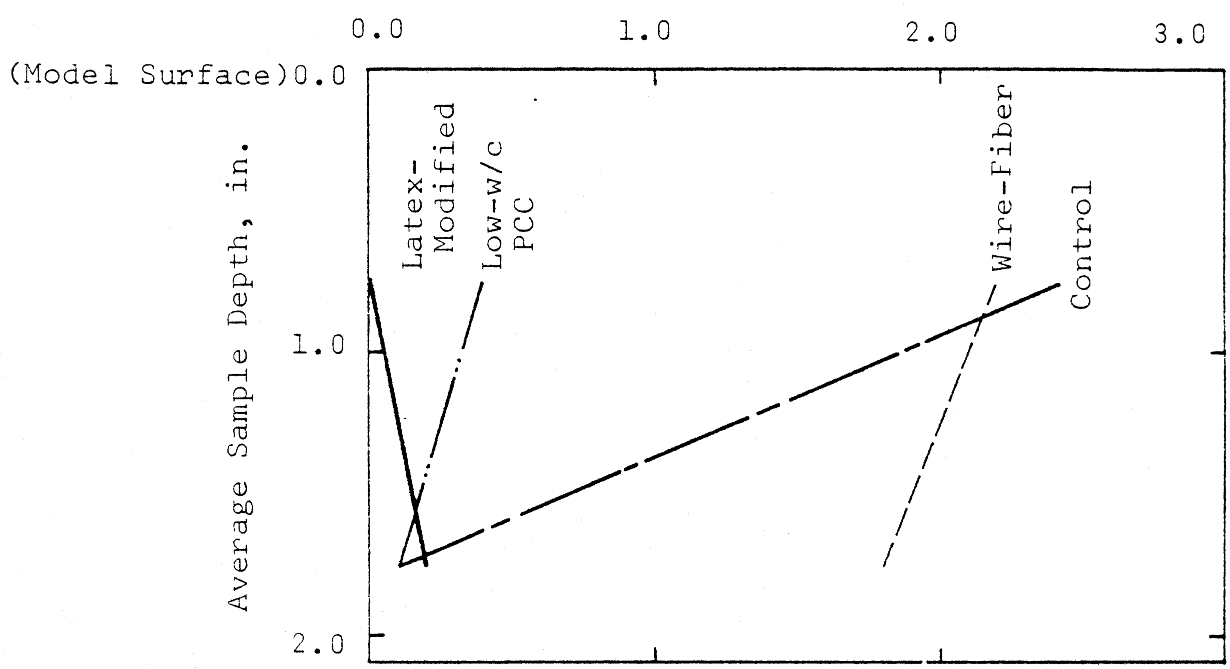


(a) Penetrated Cl^- in field structures in 1980.



(b) Penetrated Cl^- in deck models in 1980.

NOTE: $lb.Cl^-/yd.^3 \times 0.59$ - $kg\ Cl^-/m^3$

in. x 25.4 = mm

Figure 6. Chloride ion contents measured by titration analysis.

The chloride penetration data for the field structures and deck models, when viewed together, indicate longer periods of protection of the reinforcing steel from Cl^- concentrations considered detrimental to performance in the bridge spans overlaid with either latex-modified concrete or low-w/c PCC than in either the spans overlaid with wire fiber-reinforced concrete or the single-lift control spans constructed with ordinary PCC.

Electrical Potentials

The electrical half-cell potentials of the reinforcing steel in the study spans were determined on two occasions in 1975 and once in each of the surveys of the spans during 1977 and 1980. The percentages of electrical potential readings falling within each of the three ranges normally used to interpret this type of data are listed in Table 9.

The only remarkable observations made concerning the electrical potential measurements is that, as shown in Table 9, the potentials in most cases shifted back and forth between the lowest and the middle ranges, and that there is currently no indication of active corrosion in any of the experimental or control spans.

Table 9

Summary of Electrical Half-Cell Potentials of Reinforcing Steel in Study Spans for 1975, 1977, and 1980

Span	Lane	Percentage of Total Number of Readings — Range of Electrical Half-Cell Potentials, (-) Volt CSE											
		0.00 to 0.20				0.20 to 0.35				> 0.35			
		4/75	6/75	8/77	4/80	4/75	6/75	8/77	4/80	4/75	6/75	8/77	4/80
Single-Lift:													
Control	EBL	---	~~100~~	100	94	100	---	---	6	--	--	--	--
	WBL	---	*	100	100	97	*	---	---	3	*	--	--
Two-Course:													
Latex-Modified	EBL	100	100	100	89	---	---	---	11	--	--	--	--
	WBL	100	*	100	93	---	*	---	7	--	*	--	--
Low-w/c PCC	EBL	100	50	100	58	---	50	---	42	--	--	--	--
	WBL	100	*	83	35	---	*	17	65	--	*	--	--
Wire-Fiber	EBL	90	100	35	94	10	---	65	6	--	--	--	--
	WBL	95	*	100	48	5	*	---	52	--	*	--	--

*Readings not taken for WBL spans during 6/75 survey.

CONCLUSIONS

On the basis of information gathered in this six-year study of two-course bonded concrete bridge deck construction, observations and conclusions are made as follows:

1. The findings presented in the interim reports from this study concerning the evaluation of construction techniques and concrete characteristics are correct.

2. The skid resistance of the control and experimental spans is excellent.

3. Soundings of the control and experimental spans indicated that incipient spalling has not occurred. For the two-course spans this, as well as the examination of cores drilled during the 1980 survey, indicates that no debonding has taken place. Sonic pulse velocity measurements through the control and experimental spans also support the statement that no incipient spalling or debonding has occurred.

4. Plastic shrinkage cracks were observed in small portions of each of the overlays with the exception of the wire fiber-reinforced overlays, which had none. A relatively large amount of cracking was found on one control span and the other was free from plastic cracking. This problem is the result of improper curing practices and it is believed that both the moist-curing and membrane-curing methods can achieve satisfactory results if properly applied for both single-lift and two-course construction. A significant amount of cracks was observed on one latex-modified concrete overlay and a conclusion concerning the cause of this problem was not made.

5. The average absorption values for the latex-modified concrete and low-w/c PCC overlays were 1.1% and 5.2%, respectively, compared to an average absorption value of 6.3% for the control and wire-fiber concretes. The latex-modified concrete and the low-w/c PCC are significantly better than the control concrete as judged on the basis of absorption, because it has been suggested that bridge deck reinforcement having a 2.4-in. (61-mm) minimum cover depth of concrete with an absorption not exceeding 4.5% to 5.0% is adequately protected for periods approaching 20 years from penetrated chloride ion concentrations considered sufficient for inducing corrosion.

6. The penetrated Cl^- contents of the study concretes in the field structures and deck models indicate that long-term protection of the reinforcement from corrosion induced by chloride ions will be provided by both the latex-modified concrete and the low-w/c PCC overlays, but not by the wire-fiber concrete.

7. The electrical potentials of the reinforcing steel indicate no active corrosion in either the two-course or control spans.

8. An attempt to verify the bridging action of polymer microfibers across microcracks in the latex-modified concrete from micrographs produced with a scanning electron microscope was not successful. The low absorption values suggest, however, that the polymer is performing as intended.

RECOMMENDATIONS

1. The condition and performance of the study spans warrant the use of two-course bonded bridge deck construction for the —

   a. placement of a protective overlay concrete above the reinforcing steel where needed to minimize the penetration of corrosive substances, while allowing less costly ordinary concrete to be used for the bulk of the deck;

   b. isolation of common construction defects, such as cracking due to subsidence and plastic shrinkage, in the base layer concrete.

   c. provision of an option to use less costly aggregates, manufactured sands that polish and fail to maintain adequate skid resistance, or natural sands with high void percentages that require more mixing water and result in less durable concrete in the base layer where they are acceptable; and

   d. improvement in the control of the depth of cover for the topmost reinforcing steel.

2. The latex-modified and low-w/c concretes exhibited improved resistance to Cl^- penetration as compared to ordinary PCC, and these should be considered acceptable as protective systems for reducing the rate of corrosion of bridge deck reinforcing steel. The wire-fiber reinforced concrete did not exhibit such resistance, however, and should not be considered an acceptable protective system.

3. The continued use of polymer-modified concretes would suggest the need for convenient and reliable test procedures for quality assurance. In Virginia the accessibility of an electron microscope would allow the convenient use of micrographs, with no investment in equipment, as part of a quality assurance program. Additional research should be conducted to determine if such a program is feasible. Also, installations of latex-modified concrete should be monitored during and after construction to determine the particular conditions that may predispose them to cracking.

ACKNOWLEDGEMENTS

This report was completed under the supervision of H. H. Newlon, Jr., director of the Virginia Highway and Transportation Research Council. The study was financed with HPR funds administered by C. L. Chambers and K. C. Clear of the FHWA. Technical advice and guidance were provided by the Concrete Research Advisory Committee, the current membership of which is listed on page ii of this report.

Technical assistance and coordination of activities throughout the study have been provided by Department employees too numerous to mention, considering both the risk of omission and the fact that changes in personnel have occurred during the years. Gratitude is expressed to those individuals by noting the following organizational units of the Department in which they have served: the Bridge, Materials, and Construction Divisions of the Central Office; the Bridge, Materials, Equipment and Survey sections of the Staunton District Office; the Luray and Edinburg Residencies; and the Berryville Area Headquarters. Special thanks are extended to the individuals who cheerfully shared the frustrations engendered by traffic, equipment, and weather.

The responsibility for performing tests and gathering data throughout the study was primarily that of C. E. Giannini, Jr. and B. F. Marshall of the Research Council staff, and their cooperation and efficiency is gratefully acknowledged. The typing of the draft report and related documents throughout the study was admirably performed by A. M. Fewell.

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