tion of the first ponding test, the specimen will be subjected to 10 cycles of the full temperature range prescribed. The specimen will remain at each temperature extreme for 30 min between cycles with the gap between slabs set at the appropriate dimension.

4. Splice test: Specimens of segmented systems will be tested in tension to rupture. The splices will be expected to be nearly as strong as an individual segment.

5. Bolt torque dissipation: In this test, anchor bolt systems will be tested for torque retention after a period of sustained vibrations. The specimen will be subjected to downward blows of 8000 lb applied in a rapid sequence to each side of the joint, which simulates high-impact wheel passages. After 50 000 consecutive blows, the torque will be tested and expected to be within 1 percent of the initial value. Also, the rigors of the test are not to have caused damage to, or loosening of, other parts.

6. Vehicular braking/traction test: A standard truck tire mounted on an axle and loaded with 8000 lb will be drawn across the specimen with the wheel locked, and then rolled back. This cycle shall be repeated 50 000 times with a period of 2 s.

7. Membrane puncture: This test is designed to determine a strip seal's capacity to resist puncture and pullout from retainers when the seal may be covered with granular debris. A rubber-tired ram apparatus will deliver 8000-lb downward blows at 5-s intervals to the center of a plate that covers the specimen.

8. Debris-expelling test: The seal will be examined after being loaded with granular debris and cycled 25 times between full-rated opening and 1.25 in for its ability to expel the entrapped debris.

9. Postinstallation testing: The only test proposed after construction is a water-ponding test for leakage. In this test, dikes will be constructed across the roadway and sidewalks, and water delivered through an unnozzled hose at the rate of 1 gal/min will flow continuously down the parapet face and across the sidewalk and curb face of the joint. The flow and ponding will continue for a period of 9 h, after which the underside of the entire joint will be examined for leakage.

CONCLUSIONS

In summary, then, observations of many installations over a significant time period lead us to conclude that, for the most part, proprietary bridge deck joint-sealing systems have not been performing as expected. They have been more successful in theory than in actual practice, where they are subjected to the rigors of traffic and the occasional unpredictable behavior of bridge structures. Ironically, the most important attribute for which these systems were designed--watertightness--has been the area in which most of the complaints originated.

Judging from the erratic performance of identical joint assemblies in different projects, the quality of workmanship in installing these sophisticated assemblies must play a critical role in their success or failure.

In today's competitive market, with some highway agencies specifying joint assemblies by name, there is little incentive for manufacturers to augment their design with any cost feature that would price them out of the market. If all joint products were held to a common set of performance ground rules, manufacturers would not only strive to improve their systems in order to comply, but they would be competing on an equitable basis and agencies would be afforded reasonable assurance of good performance at competitive prices. An effective performance specification, geared to stipulating desirable attributes verifiable to some extent by meaningful testing programs before and after installation, should be the solution to this problem.

Contractual restraints and practical considerations are such that the detection of those systems with desirable attributes, and elimination of those without, must be performed before an extensive in-service period. Consequently, it is only through a properly designed qualification program that the concept of a performance specification can be acceptable to all parties.

An important area for additional research concerns the further development of test criteria that can truly gauge a proprietary sealing system's quality. Judgment only has served to compile the criteria presented in this report. More empirical data are needed to equate the form, magnitude, and cyclical variation of laboratory-applied loads with the actual lifetime experience of a sealing system.

REFERENCE

1. Howard, Needles, Tammen & Bergendoff. Bridge Deck Joint-Sealing Systems: Evaluation and Performance Specification. NCHRP, Rept. 204, 1979, 46 pp.

Publication of this paper sponsored by Committee on Sealants and Fillers for Joints and Cracks.

Thermal Compatibility of Thin Polymer-Concrete Overlays

MICHAEL M. SPRINKEL

Thin polymer-concrete overlays that provide low permeability and high skid resistance can be installed on bridge decks with minimal disruption to traffic and at about one-half the cost of alternative service-life-extending measures such as portland cement concrete overlays. Unfortunately, laboratory tests have indicated that the temperature changes to which bridge decks are typically subjected are sufficient to cause deterioration and eventual failure of the overlays placed in Virginia. The deterioration is caused by the development of stress in the bond between the concrete and overlay that results from differences in the moduli of elasticity and the coefficients of thermal expansion of the two materials. Thermally induced cracks have been noted in the overlay, the base concrete, and the bond interface—a majority of them in the medium least able to withstand the stress. Cracks in the overlay increase its permeability, and cracks in the base concrete or the bond interface lead to delamination of the overlays. It is estimated that a properly installed overlay prepared with either of the two polyester resins tested to date in Virginia will have a useful service life of at least five years, which, considering its ease of application, may be acceptable for bridges where it is difficult to close a lane to make a more permanent repair. A longer service life should be possible if more flexible resins are developed.

Figure 1. Resin is sprayed over deck surface.

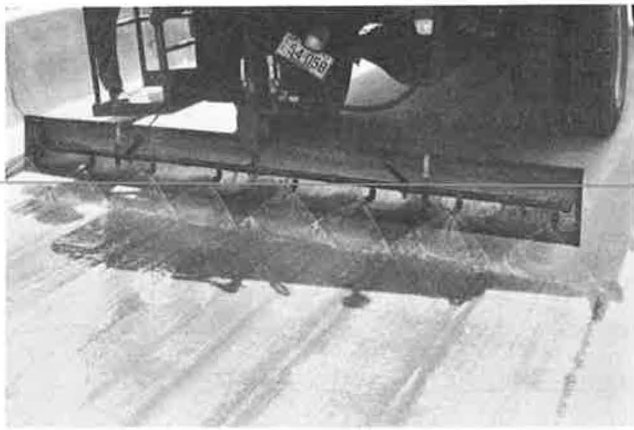


Figure 2. Silica aggregate is broadcast over resin.

Thin polymer-concrete (PC) overlays have been installed on portland cement concrete bridge decks in several states during the past four years (1). This type of overlay was developed by the Brookhaven National Laboratory for the Implementation Division of the Office of Development, Federal Highway Administration (FHWA), as a protective system for bridge decks (2). It consists of four layers of polymer--a hard, glossy solid commonly called plastic--and clean, dry, angular-grained silica sand in a total thickness of 0.5 in applied to the top of a portland cement concrete deck to provide a relatively impermeable, skid-resistant wearing surface. Typically, the resin is sprayed uniformly over the surface of the deck and is covered by fine aggregate broadcast over it, as shown in Figures 1 and 2. Following polymerization, the excess aggregate is removed with a vacuum device prior to the placement of a subsequent layer. The principal advantage of the overlay over other protective systems is that it can be installed without the removal of a large amount of concrete and with minimal disruption to traffic, approximately 8 h/lane for a 250-ft bridge: 4 h for deck preparation and 1 h/layer to install the polymer.

PC overlays were installed by a subcontractor on five bridges near Williamsburg in 1981 and by state maintenance forces on a sixth bridge on Beulah Road in northern Virginia in 1982. This paper is based on the laboratory work conducted as part of the evaluation of the installation and performance of these overlays.

OBJECTIVE

When two materials of different properties are bonded together, a shearing stress develops when the composite is subjected to a change in temperature. Van Vlack's equation (3, p. 361) for this stress is

$$S = [(C_p - C_c)\, E_p E_c\, \Delta T]/(E_p + E_c) \qquad (1)$$

where

S = shearing stress (lb/in²),
C_p = coefficient of thermal expansion for PC overlay (in/in/°F),
C_c = coefficient of thermal expansion for portland cement concrete (in/in/°F),
E_p = modulus of elasticity of PC overlay (lb/in²),
E_c = modulus of elasticity of portland cement concrete (lb/in²), and
ΔT = temperature change (°F).

The shearing stress is a function of the temperature change and the coefficient of thermal expansion and moduli of elasticity of the two materials. A shear failure in the vicinity of the interface can be expected if the shear stress exceeds the shear strength of either of the materials or the strength of the bond. The objective of the research reported here was to gain an indication of the thermal compatibility of PC overlays and portland cement concrete bridge decks. Conclusions are based on a comparison of the theoretical shear stress as reflected by the Van Vlack equation and the measured strength and observed performance of specimens subjected to cycles of temperature change.

TESTING PROGRAM

Materials

The resins used were clear, low viscosity, highly resilient, general purpose, unsaturated polyester resins. With a viscosity of 100-150 cP at 77°F and a density of 9.08 lb/gal, the resins were U.S.S. Chemical's blend LB183-13 and Reichhold Chemical's blend Polylite 90-570. The first course contained 1 percent of Union Carbide's A-174 coupling agent and 1 percent of Air Products and Chemicals, Incorporated's, Surfynol S440 wetting agent to enhance the bond strength and reduce surface tension. The second, third, and fourth courses contained 0.5 percent of A-174 and 0.5 percent of Surfynol S440 (4).

Two initiators were used. One was 60 percent methylethylketone peroxide (MEKP) ($C_4H_8O_2$) in dimethyl phthalate. The other was 40 percent benzoyl peroxide dispersion (BPO-40), which is equal to Reichhold Chemical's formulation 46-742.

Also, two promoters were used. One was approximately 6 percent active cobalt in naptha (CoN). The second was N, N, dimethyl aniline (DMA) ($C_6H_4N[CH_3]_2$).

The aggregate was a clean, dry (less than 1 percent moisture), angular-grained silica sand free of dirt, clay, asphalt, or other organic materials, and conforming to the following gradations:

	Percentage Passing Sieve No.				
Layer	8	12	16	20	30
3 and 4 (top)	95-100	40-55	5-15	Max. 1	---
1 and 2 (bottom)	---	95-100	20-55	5-10	Max. 1

The coarse aggregate was used in the top two layers to provide skid resistance.

Fabrication and Testing of Overlays

In the laboratory, 2.00 lb/yd^2 of resin were applied for the first layer and 2.75 lb/yd^2 for subsequent layers. The overlays were prepared at a temperature of 74°F. The application rates in the field were similar, but some of the installations were made at temperatures between 65°F and 100°F. Gel times of 10-20 min were typical for the laboratory and the field. The principal difference between the overlays was the quantity of aggregate used, although other differences included the brand of polyester resin, the type of initiator and promoter, and the method of fabrication. For the laboratory specimens, the application rate for the aggregate ranged from 0 to 21 lb/yd^2 per layer (0 to 84 lb/yd^2 for four layers), and for the field applications it ranged from 33 to 70 lb/yd^2 for four layers. Because it was impossible to retain more than about 50 lb/yd^2 of aggregate in the 10.25 lb/yd^2 of resin used in the four layers, at the high aggregate application rates the excess aggregate was removed prior to placing a subsequent layer.

Some of the overlays were prepared by using CoN at a dosage of 0.5 percent by weight of resin and MEKP at a dosage of 1.2 percent, and others were prepared with BPO-40 at a dosage of 3.0-6.0 percent and DMA, which was added by the manufacturer of the resin, at a dosage reported to be 0.4 percent.

Thirty-six specimens of the PC overlay materials, which measured 3 in wide by 11 in long by 0.3-0.6 in thick, were prepared for determinations of density, moduli of elasticity, and coefficient of thermal expansion (unbonded specimens). In an additional six specimens prepared for the same purpose, the PC overlay was placed on portland cement concrete without sandblasting the concrete surface (bonded specimens). Prior to any measurements, the six bonded overlays were removed from the base concrete by subjecting the composite to a freeze-thaw test in accordance with procedure A of ASTM C666. In addition, PC overlays were placed on the sandblasted tops and bottoms of eighteen 3x4x16-in portland cement concrete specimens that were then subjected to cycles of temperature change. The portland cement concrete had a 28-day compressive strength of 5150 lb/in^2, a shear strength of 612 lb/in^2, and a plastic density of 145 lb/ft^3, which are reasonably typical of values for bridge deck concrete. Also, twenty-three 4-in-diameter cores removed from six bridge decks overlaid with PC were tested to determine the effect of thermal cycles on the permeability of the overlays. In addition, twenty-two 2.75-in-diameter cores were removed from an overlaid deck and thirty-four 4.00-in-diameter cores were removed from five bridges that were candidates for overlays. After PC overlays were placed on the 4.00-in cores in the laboratory, all cores were subjected to a shear test to determine the effect of thermal cycles on the shear strength of the bond between the PC overlay and the base concrete. Finally, twenty-two 1.00-in-diameter cores were removed from an overlaid bridge and tested to determine the effect of thermal cycles on the tensile strength of the bond.

Thermal Cycles

The thermal cycles were applied at a rate of three per day and the temperature ranged from a low of 10°F to a high of 100°F. This range was selected because it was believed to approximate the high and low temperatures that would be encountered in the field. It is only slightly higher than the -6°F to 77°F specified by ASTM C884-78 (Thermal Compatibility Between Concrete and Epoxy-Resin Overlay), but the rate of cycling is six times faster than the one cycle per two days specified. The test was easily applied with equipment on hand and provided results in a short time and at a reasonable cost. Based on tests after one year of service life, it is estimated that in the first year an overlay is subjected to the equivalent of 50 cycles. The use of ASTM C884, which requires the use of 12x12x3-in specimens, should not lead to significantly different conclusions concerning the thermal compatibility of PC overlays and concrete.

RESULTS

Dynamic Moduli of Elasticity

Figure 3 shows the dynamic moduli of elasticity (E_p) of the PC overlays based on ASTM C215-60 as a function of the aggregate application rate. It can be seen that, as the rate increased, the E_p increased up to a maximum for a rate of 52 lb/yd^2 and then tended to level off or decrease. Also, the moduli of elasticity for LB183 appeared to be higher than that for 90-570 for the unbonded specimens. Except when no aggregate was applied, the lowest E_p for a given aggregate application rate was found for the bonded LB183 overlays. The reader is cautioned to remember that the bonded LB183 overlays were fabricated in bond to concrete but were not bonded when the moduli were measured. Because the specimens prepared with no aggregate were very thin and flexible, it was difficult to accurately determine their E_p. The omission of aggregate from one or two layers produced moduli less than that found when one-fourth of the total amount of aggregate was applied to each of four layers.

Coefficients of Thermal Expansion

Metal studs were installed in the ends of the 36 unbonded and 6 bonded specimens of PC overlay and the 6 specimens of base concrete to allow a determination of the length of the specimens at a given temperature. The length of each specimen was determined three times at each of three temperatures: at 0°F, as provided by a freezer; at 74°F, as provided by the laboratory atmosphere; and at 140°F, as provided by a water bath. Plastic bags were used to keep the specimens from absorbing moisture when placed in the baths. Based on the length measurements, the coefficients of thermal expansion were determined for the 74°F base temperature.

Figure 4 shows the coefficient of thermal expansion (C_p) as a function of the aggregate application rate for four layers. The C_p decreased as the rate increased up to a minimum value for a rate of 52 lb/yd^2, then tended to level off. The two products did not appear to have significantly different values, although the C_p for 90-570 was slightly lower at an aggregate application rate of 52 lb/yd^2 or more. The lowest C_p's were found for the bonded LB183 specimens. The omission of aggregate from two layers produced C_p's that were only slightly higher than that found when one-fourth of the total weight of aggregate was applied to each of four layers. The portland cement concrete base had a C_p of 5.7×10^{-6} in/in/°F.

Theoretical Shear Stress

Figure 5 shows the theoretical shear stress in the vicinity of the bond interface as a function of the aggregate application rate based on the Van Vlack equation and the values for moduli of elasticity and coefficients of thermal expansion determined for the base concrete and the PC overlays. A value of 4.2×10^6 lb/in² as determined by ASTM C215 was used for the modulus of elasticity of the base concrete. The use of other values that might be found for bridge deck concrete would not have a significant effect on the theoretical shear stress.

It can be seen from Figure 5 that the shear stress for a 1°F change in temperature decreased as the aggregate application rate increased beyond 7 lb/yd². The highest stress occurred for an aggregate application rate of 4 to 7 lb/yd², depending on the product. The theoretical shear stress, based on values of E_p and C_p, for the unbonded specimens was low at an aggregate application rate of zero because the E_p was low. Also, the theoretical shear stress was less for 90-570 than for LB183

Figure 3. Modulus of elasticity as function of sand application rate.

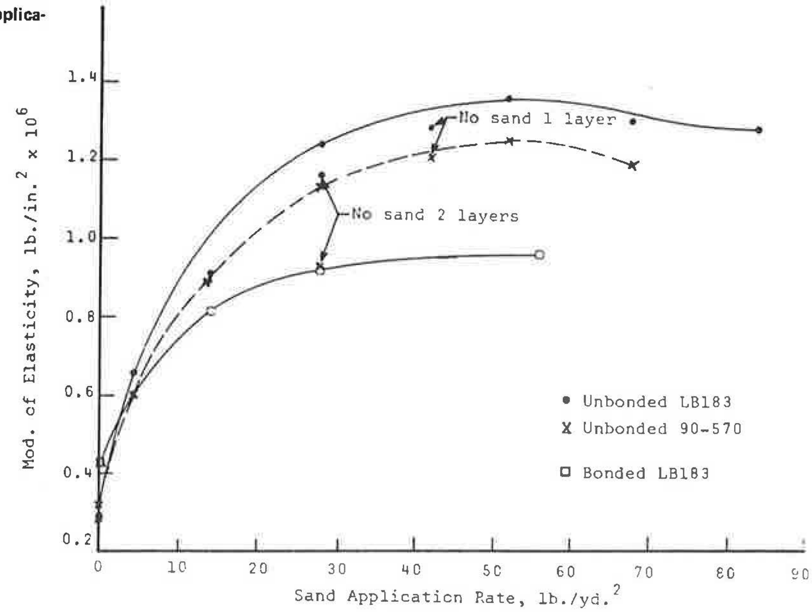


Figure 4. Coefficient of thermal expansion as function of sand application rate.

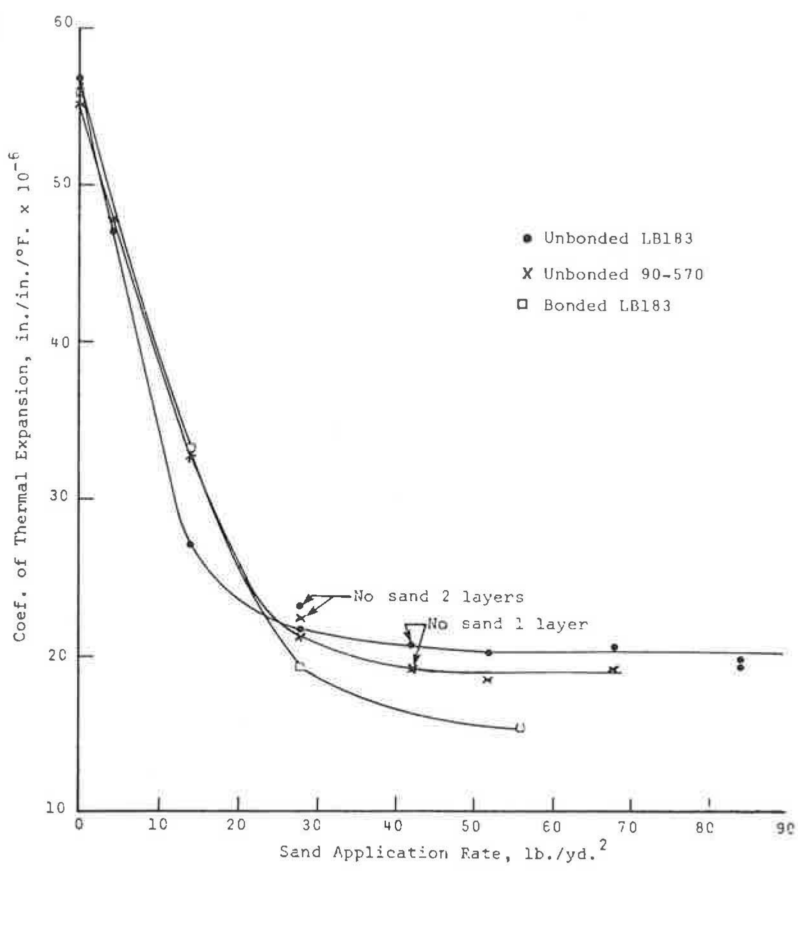


Figure 5. Shear stress/°F as function of sand application rate.

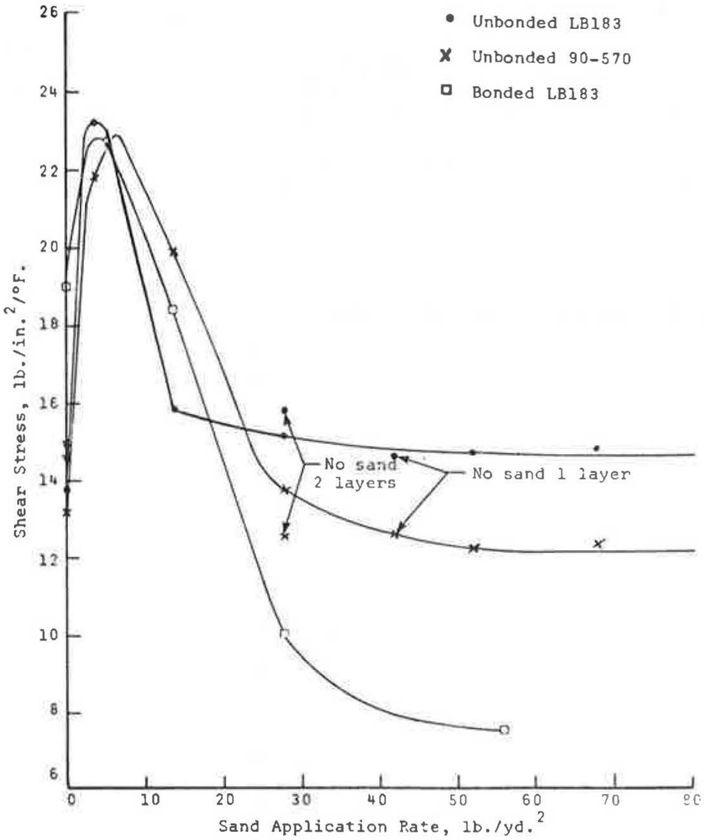
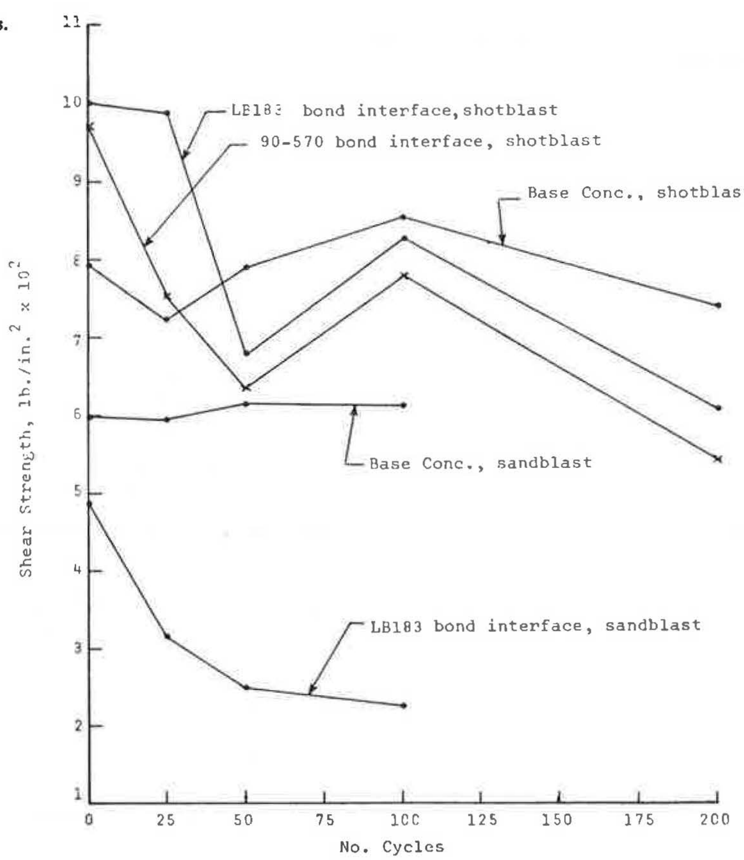


Figure 6. Shear strength as function of number of thermal cycles.



for the 52-lb/yd² sand application rate. The lowest theoretical shear stress was found by using values of E_p and C_p found for the bonded LB183. The fabrication of the bonded overlays or the fatigue that resulted from the freeze-thaw test used to remove the overlay caused the E_p and C_p to be lower than those found for the unbonded specimens. The E_p and C_p appear to have changed as a result of the overlay being placed on the concrete and therefore to have taken on properties more compatible with those of the base concrete and properties that provide for less shear stress for a given temperature change. Although E_p decreased and C_p increased, on average, the omission of aggregate from one or two layers of the unbonded specimens prepared with an aggregate application rate of 14 lb/yd² per layer did not produce a theoretical shear stress that was significantly different from that produced when one-fourth of the total amount of aggregate was applied to each of four layers.

Shear Strength

To obtain an indication of the shear strengths of the portland cement concrete and the concrete and PC overlay composites, cores were subjected to two shear tests. For the first test, the force was directed through the bond interface; for the second, it was directed through the concrete at approximately 2.5 in below the bond interface, which provided an indication of the shear strength of the portland cement concrete. Prior to the tests, the cores were subjected to from 0 to 200 thermal cycles, as described earlier.

A plot of the shear strength of the bond interface and the base concrete as a function of the number of thermal cycles is shown in Figure 6 for twenty-two 2.75-in-diameter cores, which had been removed from the Beulah Road bridge two weeks after it was overlaid, and for thirty-four 4.00-in-diameter cores removed from five bridges in Virginia considered as candidates for PC overlays. The top surfaces of the 4.00-in cores were sandblasted and LB183 resin overlays were placed on them in the laboratory. The deck of the bridge on Beulah Road had been shotblasted before the LB183 and 90-570 overlays were installed.

It can be seen that the shear strength of the portland cement concrete did not change as a result of the thermal loading. On the other hand, the shear strength of the bond interface decreased as the number of thermal cycles increased. For the shotblasted deck, the shear strength of the bond interface was higher than that of the base concrete at zero thermal cycles, whereas for the sandblasted cores it was lower. Also, it can be seen that the shear strength of the bond interface was consistently higher for LB183 than for 90-570, but this may have been due to factors other than these products. It was felt that the shotblasting removed more concrete from the surface that received the LB183 resin than from the surface with the 90-570 resin. For the sandblasted cores, the low strength of the bond interface at zero cycles may be attributed to a poor job of preparing the concrete surface prior to placing the overlay. Evidently, sandblasting cannot be depended on to adequately prepare the surface, and shotblasting is considerably more dependable.

The location of the failures that occurred during the shear tests provided further evidence that the strength of the bond interface deteriorated when subjected to cycles of temperature change. The table below shows that, for the shotblasted surfaces at zero thermal cycles, all of the shear failures involved only the base concrete, whereas with the sandblasted surface many of the failures involved the bond interface. The number of shear failures at indicated locations is as follows:

No. of Thermal	Shotblasted Surface			Sandblasted Surface		
Cycles	Bond	Concrete	Both	Bond	Concrete	Both
0	0	8	0	1	5	8
25	1	0	2	4	0	2
50	2	0	2	6	0	4
100	3	0	2	2	0	2
200	2	0	0			

At 25 or more thermal cycles, all of the failures for both types of surfaces involved the bond interface and none involved only the base concrete.

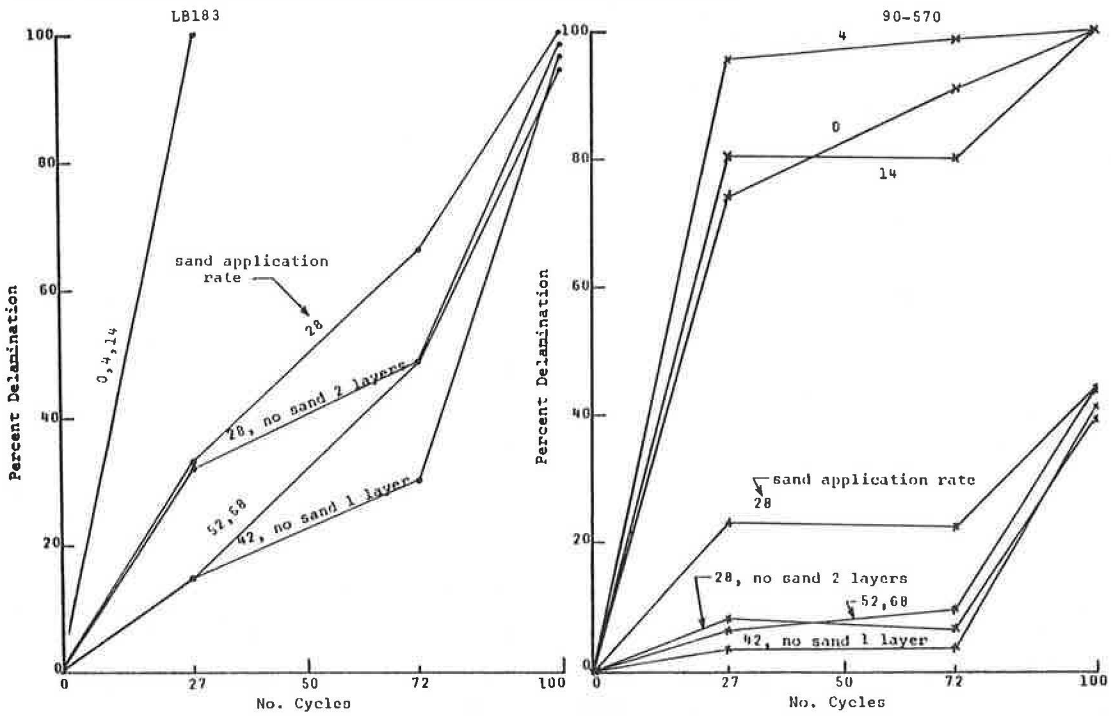
Based on the curves in Figure 5, the thermal cycles produced a maximum shear stress at the bond interface of 480 to 934 lb/in². The magnitude of the maximum shear stress is based on a sand application rate of 56 lb/yd² and a maximum temperature differential of 64°F, which comes from an equilibrium fabrication temperature of 74°F and a minimum temperature of 10°F. Cycles of stress of this magnitude should reduce the shear strength of the bond interface rapidly. If it is assumed that 50 cycles equals one year of service life, at one year a sandblasted surface would provide inadequate bond strength and a shotblasted surface would provide a strength slightly less than that of the base concrete. A shotblasted surface would continue to provide acceptable bond strength for more than four years (200 cycles). The deterioration in shear strength that results from the thermal cycles and shown in Figure 6 is evidence that the theoretical shear stresses shown in Figure 5 are reasonable and capable of causing eventual failure of the bond interface.

Delamination

To gain an indication of the amount of cracking that would occur at the bond interface as a result of thermal loading, PC overlays were placed on specimens that have a surface area of 3x16 in. The surfaces were sandblasted prior to placing the overlays. Figure 7 shows the percentage of the perimeter of the bond interface shown by a visual inspection to contain cracks as a function of the number of thermal cycles. It is obvious from the figure that thermal loading initiated cracks at the bond interface in all the specimens. The percentage of delamination, as indicated by the length of the cracks, increased with an increase in the number of thermal cycles. More delamination occurred with the LB183 resin than with the 90-570 for the same number of cycles and the same aggregate application rate, but this may be related to the condition of the surface of the portland cement concrete. The LB183 overlay was placed on the screeded tops of the specimens and the 90-570 overlay on the molded bottoms. Either the bottoms of the specimens received a better cleaning than the tops or the tops needed more cleaning than the bottoms. The data in Figure 7 support the need to remove unsound or weak concrete prior to placing the overlays.

More important, the data in Figure 7 show the effect of the aggregate application rate on the amount of delamination. It can be seen that, for application rates of 0-14 lb/yd², PC overlays approached complete delamination in 27 cycles or less. At an application rate of 28 lb/yd², the amount of delamination was reduced significantly, and rates in excess of 28 lb/yd² generally resulted in the least amount of delamination. It is interesting to note that for overlays prepared with

Figure 7. Delamination versus number of thermal cycles for different sand application rates.



an application rate of 14 lb/yd² per layer, the omission of aggregate from one or two layers did not have a significant effect on the amount of delamination, which would be expected based on the shear-stress data in Figure 5. The delamination data in Figure 7 are further evidence that the theoretical shear-stress data reported in Figure 5 are reasonable, particularly the curve for the bonded specimens. The data in Figure 7 also indicate that, even at the higher aggregate application rates, when the base concrete is sandblasted, a major amount of delamination can be expected after two years of service life (100 cycles).

Tensile Strength of Bond Interface

Twenty-one 1-in-diameter cores were removed from the bridge on Beulah Road and subjected to a tensile test in which the overlay was pulled from the base concrete. Prior to the test, eight of the specimens were subjected to 50 thermal cycles. The tensile strength data for 0 and 50 cycles are shown in the table below:

	0 Cycles		50 Cycles	
Item	LB183	90-570	LB183	90-570
Strength (lb/in²)	337	268	210	198
No. of failures				
Polymer	0	0	0	0
Bond	2	0	3	1
Concrete or aggregate	7	4	1	3

On average, a 26-38 percent reduction in the strength of the composite resulted from the 50 thermal cycles. Tensile strength reductions of up to 27 percent were noted for cores taken from the bridges near Williamsburg following one year of service life.

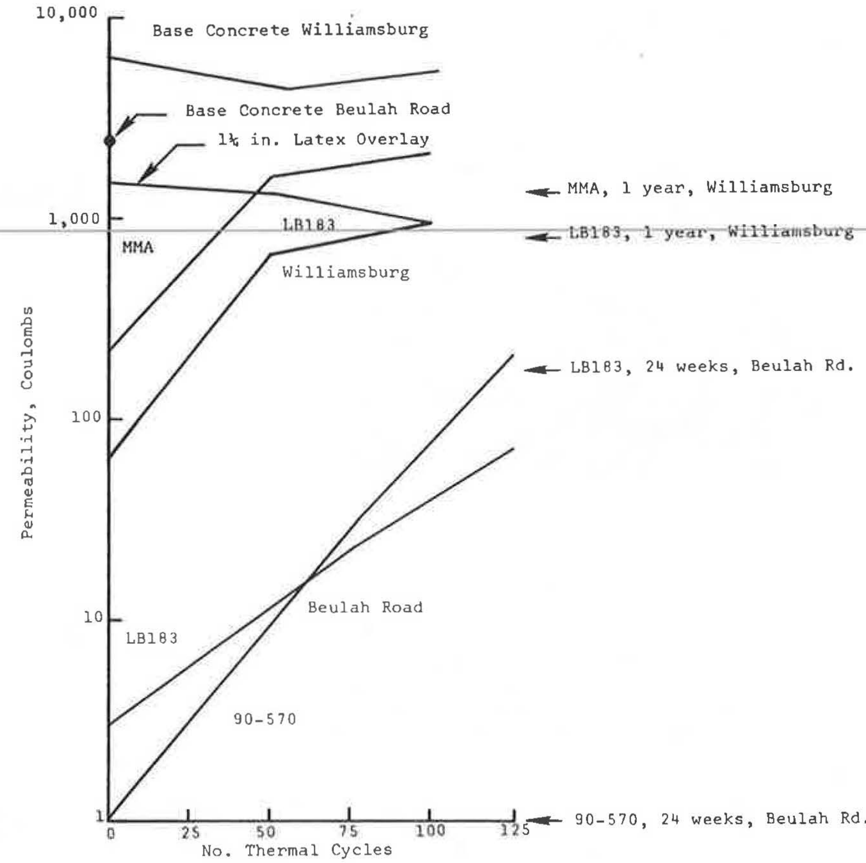
As can be seen from the table above, a bond failure occurred only 15 percent of the time for 0 cycles, whereas after 50 cycles 50 percent of the failures were in the bond. A similar pattern of failure was noted in the cores taken from the bridges near Williamsburg, where 44 percent of the failures were in the bond for cores taken following the placement of the overlays and 75 percent of the failures were in the bond for cores taken after one year of service life. At 0 cycles, tensile strength was primarily a function of the strength of the aggregate and concrete, whereas after 50 thermal cycles the bond had been weakened to the point that it was of about the same strength as the concrete and aggregate. The deterioration in the tensile strength of the bond interface with thermal cycles was similar to the deterioration in the shear strength.

Permeability

The PC overlays are placed on bridges to provide a protective layer that will be less permeable than the deck concrete so as to curtail the infiltration of chlorides and water. To obtain an indication of the chloride permeability of the overlays and the effect of thermal loading on chloride permeability, twenty-three 4-in-diameter cores removed from six bridges in Virginia were tested by using the rapid permeability test recently developed by the Portland Cement Association for FHWA (5). All the cores were tested as removed from the bridges and again after being subjected to a number of thermal cycles. Although the developers of the test recommended that specimens not be tested a second time, the experience from this study indicates that the average permeability of four specimens with PC overlays does not change more than 5 percent as a result of conducting the second test.

Figure 8 shows the relation between chloride permeability and the number of thermal cycles based on the average of the logarithm of the permeabilities of the specimens. There it can be seen that the permeability of the overlays was very low initially but increased after the overlays were subjected

Figure 8. Permeability of PC overlays to chloride ion as function of number of thermal cycles.



to thermal cycles. After 100 cycles, estimated to represent two years of service life, the average permeability of the LB183 resin used near Williamsburg was approaching that of a latex overlay, but much less than that found for concrete. The average permeability of the LB183 and 90-570 resins used on Beulah Road was an order of magnitude lower. The worst deterioration with thermal loading was found for a PC overlay constructed with methyl methacrylate (MMA). For this overlay, the permeability was equal to that of a latex-modified concrete overlay after only 50 cycles, but after 100 cycles was less than that of the base concrete.

The permeability of cores removed from the bridges in Williamsburg after one year of service life and the bridge on Beulah Road after 24 weeks of service life are also shown in Figure 8. The data suggest that one year of service life is comparable with 50-75 thermal cycles. The data in Figure 8 suggest that the polyester overlays will provide a permeability less than that of the base concrete for at least five years.

Evidently, the stress induced by temperature changes that causes a deterioration in the strength of the bond interface also causes microcracking in the PC overlay that allows infiltration of chlorides. The microcracking is probably the greatest in an overlay where the bond and concrete strengths are high and therefore less likely to fail and relieve the stress in the overlay.

Figure 9 shows an extreme case of microcracking in a bonded overlay. The specimens, pictured following one thermal cycle conducted in accordance with ASTM C884-78, were prepared with MMA resin and aggregate application rates of 0, 14, 28, and 56 lb/yd². Evidently, the shear strength of the bond interface exceeded the tensile strength of the overlay with no aggregate, and the overlay, therefore, cracked. Because the failure occurred after 1 thermal cycle, the stress was relieved and there was no failure of the bond after 20 cycles.

Although the specimen with no aggregate in Figure 9 represents an extreme case of cracking in a PC overlay, the same type of cracking occurred to a lesser extent in overlays that contained aggregate. The cracks were not as visible, but the permeability data confirmed their presence.

Curing shrinkage probably contributes to the incidence of cracking, and the amount of shrinkage decreases as the aggregate content increases. At a sand application rate of 56 lb/yd², LB183 and 90-570 had 24-h shrinkages of 0.2 and 0.1 percent, respectively.

FAILURE OF PC OVERLAYS

The data clearly indicate that the temperature changes to which bridge decks are typically subjected are sufficient to cause a level of stress that can lead to the deterioration and eventual failure of PC overlays. The failures can be grouped into one or more of three basic types, as follows:

1. The formation of vertical cracks through the thickness of the overlay (Figure 9),
2. The shearing of the portland cement concrete below the bond line (Figure 10), and
3. The deterioration of the bond between the PC overlay and the base concrete.

The formation of vertical cracks increases the permeability of the overlay and reduces its effectiveness in preventing the infiltration of chlorides. It will be the predominant mode of failure

Figure 9. Top view of bonded PC overlays prepared with MMA and aggregate application rates of 0, 14, 28, and 56 lb/yd^2 (from bottom to top).

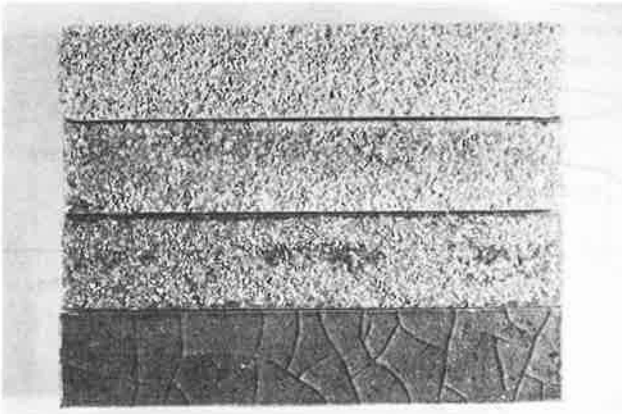
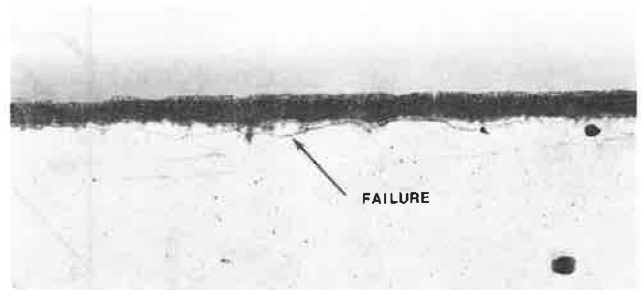


Figure 10. Shear stress failure of portland cement concrete below bond surface.



on bridges where the shear strength of the base concrete and the bond strength are high or the modulus of elasticity of the overlay is high or the tensile strength is low. The failure will likely occur after a few cycles of temperature change. The overlay will likely remain bonded to the base concrete until freezing and thawing action causes delamination.

The shearing of the concrete below the bond line causes the PC overlay to delaminate with concrete remaining bonded to its underside, as shown in Figure 10. The failure is most likely to occur when the shear strength of the base concrete is low and the bond is good and the tensile strength of the overlay is high. The failure will likely occur after a few cycles of temperature change and result in the delamination of the PC overlay.

The deterioration of the bond between the PC overlay and the base concrete causes the overlay to delaminate with no concrete remaining on the underside. The failure is likely to occur when the surface preparation prior to the installation of the overlay is poor or when the shear strength of the base concrete and the tensile strength of the overlay are high. Where the initial bond is poor, the failure will likely occur after a few cycles of temperature change (less than 100 cycles or two years of service life). Where the initial bond is good, a significant number of thermal cycles may be required to complete the failure (more than 250 cycles or five years of service life).

COST AND SERVICE LIFE

Based on data compiled in May 1982, the cost of PC overlays is estimated to be between \$20 and \$25/yd^2, as seen in the table below:

	Cost (\$/yd^2)	
Activity	0.5-in Polymer	1.25-in Latex
Deck preparation	4	9
Overlay installation	15-20	20-25
Traffic control	1	5-20
Total	20-25	34-54

The cost of the materials is approximately \$11/yd^2 and that for shotblasting is approximately \$4/yd^2, depending on the size of the job. Although overlays were installed on four bridges in Virginia in 1981 at a cost of \$41/yd^2, it is believed that with more competition, or with the use of the state's forced labor, the cost of labor can be from \$4 to \$9/yd^2. The cost of traffic control is negligible.

The cost of an alternative type of deck repair that requires the scarification and removal of the top 0.50 in of old concrete and the installation of a 1.25-in-thick overlay of low permeability portland cement concrete that contains a latex modifier is estimated to be between \$34 and \$54/yd^2. These figures assume a cost of \$9/yd^2 for removing the old concrete, \$20-\$25/yd^2 for the installation of the overlay, and \$5-\$20/yd^2 for traffic control. Clearly, the PC overlay has the most obvious cost advantage over latex-modified concrete when traffic-control costs are high, such as in high-traffic-volume situations where it is difficult and expensive to close a lane to traffic.

From the information at hand, it is reasonable to estimate that a PC overlay will provide a skid-resistant wearing surface of low permeability for a period of at least five years, depending on the quality of the installation and the characteristics of the weather and traffic to which the overlay is subjected. Where there are high volumes of traffic that make it difficult, if not impossible, to close a lane except for short periods of time, it is quite possible that a service life of five years or less can be economical, particularly when one considers that the overlay can be installed for about 50 percent of the cost for an alternative installation. Once the overlay fails, another could be installed. Where a lane can be closed for an extended period of time, a service life of at least 5 years might be unacceptable, since alternative and nonexperimental repairs such as full-depth concrete replacement or the removal of the top 0.5-2.0 in of concrete and the installation of a portland cement concrete overlay of low permeability, which will last for 20 years, could be more cost effective.

The engineer must make a decision whether or not to specify a PC overlay based on the characteristics of the bridge being repaired. Recognizing that a PC overlay should be considered a temporary installation, the longest achievable service life for a specified environment of temperature change would be expected from a PC overlay installed on properly shotblasted portland cement concrete that has a high shear strength, preferably in excess of 600 lb/in^2. The overlay must contain sufficient sand, preferably five times the weight of resin, and have a low enough modulus of elasticity to minimize cracking. The PC overlays are clearly best suited to extend the service life of bridge decks where traffic volumes are high and where the chloride ion content of the concrete at the level of the reinforcing steel is less than the 1.2 lb/yd^3 required to cause corrosion.

CONCLUSIONS

1. Thin PC overlays that provide low permeability and high skid resistance can be installed on bridge decks with a minimum of disruption to traffic and at about one-half of the cost of alternative service-life-extending measures such as portland cement concrete overlays.

2. Laboratory tests indicate that the temperature changes to which bridge decks in Virginia are typically subjected are sufficient to cause deterioration and eventual failure of the overlays. The deterioration results from the development of shear stress in the vicinity of the bond between the concrete and overlay because of differences in the moduli of elasticity and the coefficients of thermal expansion of the materials. The Van Vlack equation provides a reasonable indication of the magnitude of the stress.

3. The thermally induced stress can be held to a minimum by placing the overlay at 60°F to 70°F, by applying an excess of aggregate to the resin, and by specifying a resin with a low modulus of elasticity and a low coefficient of thermal expansion.

4. Thermally induced cracks have been noted in the overlay, the base concrete, and the bond interface--a majority of them in the medium least able to withstand the stress. Cracks in the overlay increase its permeability and cracks in the base concrete or the bond interface lead to delamination of the overlays.

5. The base concrete is least likely to fail when its shear strength is high, its surface is prepared by shotblasting, and an excess of aggregate is applied to the resin.

6. It is estimated that a properly installed overlay prepared with either of the two polyester resins tested in this study will have a useful service life of at least five years that, considering its ease of application, may be acceptable for bridges where it is difficult to close a lane to make a more permanent repair.

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Effects of Creep on Adhesive-Bonded Composite Concrete-Metal Deck Slabs

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In this study, the long-term effect of creep on the effectiveness of adhesives as shear connectors between concrete and corrugated steel decking in composite slabs was analyzed. Approximately three years ago, eight specimens were constructed: four control specimens with no adhesive bonding the concrete to the steel deck and four specimens that used an adhesive as a shear connector. Six of these specimens (three control and three adhesive) were tested three years ago. The remaining two slabs were loaded with a uniform load of 150 lb/ft^2 for 2.5 years. These remaining two slabs were then loaded to failure in the identical manner that the earlier six slabs were and the same parameters were measured. Direct comparison of the parameters provided insight as to the effects of this prolonged creep loading on the effectiveness of the adhesive as a shear connector. Instead of the strength loss, which might have been expected, the creep-loaded specimens actually showed a very slight gain in strength and stiffness.

The use of composite deck slabs in transportation structures is not new. Steel-concrete slabs have been used for parking structures, and, in the western United States, timber-concrete composite decks have been used successfully for short span bridges. In commercial buildings, of course, the steel-concrete composite slab is in very wide use (1).

Structurally, the steel-concrete composite slab is a very advantageous combination, since the concrete accepts the flexural compressive stress and the steel deck accepts the flexural tension stress. The capacity of the composite slab is greatly influenced by the strength of the connector used at the interface of the concrete and the steel deck. When the composite slab is subjected to flexure, a horizontal shear stress develops at the interface. The connector must be able to accept this shear stress and prevent any slippage, which would destroy the structural integrity of the composite slab. The most common type of connector is the welded stud that projects up into the concrete slab. Adhesives as shear connectors in composite construction (2) can be advantageous from two points: (a) the adhesive furnishes a continuous bonding plane, thereby eliminating shear concentrations caused by the discrete shear connectors, and (b) the adhesive eliminates the installation of separate connectors, which