INTERNALLY SEALED CONCRETE FOR BRIDGE DECK PROTECTION

Interim Report No. 1

by

Samuel S. Tyson
Research Engineer

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

Virginia Highway and 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

July 1978

VHTRC 79-R2

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SUMMARY

The study reported here was performed to extend the body of knowledge concerning the use of internally sealed concrete to protect bridge deck reinforcing steel. A laboratory determination of the properties of the wax and concrete used and a field evaluation of a method of heat treatment were made. The experimental structure was a three-span bridge on which internally sealed concrete was applied as an overlay.

The concrete mixture had good placement characteristics and resulted in good properties for internally sealed concrete. It had a water to cement ratio of 0.47, a cement content of 752 lb./yd.^{3} (446 kg/m^3), a wax bead content of 114 lb./yd.^{3} (68 kg/m^3), and included 4% to 6% entrained air. A similar mixture should be used on any internally sealed decks to be constructed. The heat treatment should be modified to prevent thermal cracking by heating entire span lengths and by heating only when ambient temperatures of 60°F (16°C) and higher have been sustained for one day. If these recommendations can be implemented, internally sealed concrete should be considered an acceptable system for protecting bridge decks.

A decision concerning the use of internally sealed concrete should not be made until further evaluations can be made of pattern cracking that has appeared to varying degrees on both the experimental and control spans.

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INTRODUCTION

The concept of internally sealed concrete was introduced by the Federal Highway Administration as a potential system for protecting the reinforcing steel in bridge decks. Internally sealed concrete is produced by adding small, spherical particles of wax to the normal constituents of a fresh portland cement concrete (PCC) mixture for placement in a deck. The wax beads are a 25/75 blend of montan and paraffin waxes and vary in diameter from 0.007 to 0.033 in. (0.18 to 0.85 mm). Subsequent to the hardening of the concrete, a heat treatment is applied to the surface of the deck to cause the wax to melt and flow into capillaries and bleed channels in the concrete matrix. Upon removal of the heat, the wax hardens in these voids to disrupt their continuity and thereby prevent access of chemicals that cause corrosion of the reinforcing steel.

PURPOSE

The purpose of this study was to extend the body of knowledge concerning internally sealed concrete by conducting —

1. a laboratory determination of the properties of the wax and concrete;
2. a field evaluation of a method of heat treatment; and
3. a comparative evaluation of deck structures with and without internally sealed concrete at the time of construction and after the first, third, and fifth winter seasons.

SCOPE

The study was conducted in the concrete laboratory at the Research Council and on a bridge construction project located on I-64 in Alleghany County. The interstate bridges included a three-span structure to receive the internally sealed concrete protective system and a parallel three-span structure containing epoxy coated reinforcing steel to serve as a control.

TESTING PROGRAM

The major properties of internally sealed concrete have been investigated by the FHWA.(1) Additional testing of this concrete was conducted to verify that the mixture components used in Virginia would result in concrete with acceptable properties and to identify areas where improvements could be accomplished.

The basic guidelines suggested by the FHWA for proportioning and mixing the concrete containing wax were observed. It was found in the laboratory, however, that the addition of small volumes of entrained air did not negatively influence any properties of the internally sealed concrete and that such additions improved certain characteristics. Therefore, in the following sections the results of tests using both non-air entrained and air entrained internally sealed concrete are reported. The testing programs for both the laboratory and field phases of the study are indicated in Table 1.

RESULTS

Properties of Wax

The specific gravity of the wax beads was determined by using a calibrated flask to measure the displacement of methanol, with a specific gravity of 0.794, by a known mass of wax. Methanol was chosen for use because of its good wetting characteristics and because it exhibited no sign of acting as a solvent on the wax. The procedure required careful manipulation of a narrow-necked flask. The test results are listed in Table 2.

The percentage of the beads containing air voids was determined to be approximately 60% by direct examination under a microscope. The magnified view of the beads is shown in Figure 1, where the air voids appear as dark spots in the spherical beads.

The melting range of the beads was found to start at 67.5°C (153.5°F) and end at 74°C (165.2°F), when the beads became completely molten. A target temperature of 85°C (185°F) was adopted for heating concrete in the laboratory and field to assure complete melting of the wax.

Table 1

Testing Program

Phase	Characteristics Measured
Laboratory	Air, slump, unit weight, temperature Wax bead characteristics (gradation, voids, and specific gravity) Wax bead distribution (solid and melted) Temperature monitoring (heat treatment) Compressive strength Freezing and thawing resistance Bond strength (to concrete) Water absorption Chloride penetration
Field	Routine acceptance tests Wax bead characteristics (gradation, voids, and specific gravity) Wax bead distribution (solid and melted) Temperature monitoring (heat treatment) Freezing and thawing resistance Water absorption Compressive strength

Table 2

Properties of Wax Beads

Specific gravity wax beads	0.856
Specific gravity melted wax	0.932
Percentage of air in beads	8.2

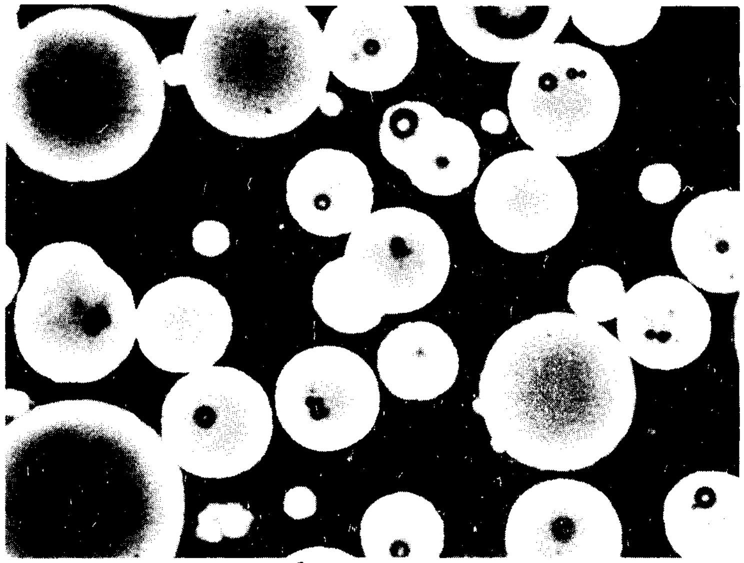

Figure 1. Spherical wax beads with air voids (dark spots). 63x

Two samples consisting of 5 grams each of wax were heated gradually to 150°C (302°F) in uncovered aluminum weighing dishes to determine if a loss of the wax material would occur during the expected heat treatment which would result in the concrete surface attaining temperatures in this range. The results, shown in Table 3, revealed insignificant losses even after 11.5 hours of heating.

A sieve analysis was performed for samples of the beads from a standard shipping drum. The results shown in Table 4 are for samples from the top and middle portions of the drum. It can be seen that the majority of the beads, approximately 95%, were retained between the number 20 and 50 sieve sizes.

Table 3

Percentage Weight Loss of Wax Beads at 150°C (302°F)

Sample No.	1 hour	3 hours	4.5 hours	11.5 hours
#1	0.2	0.3	0.5	0.8
#2	0.2	0.3	0.4	1.2

Table 4

Sieve Analysis of Wax Beads

Sieve No.	Percent Retained	Cumulative Percent
	Top of Drum	
16	0.02	0.02
20	0.04	0.06
50	94.84	94.90
80	3.74	98.64
100	0.69	99.33
200	0.63	99.96
pan	0.04	100.00
	Middle of Drum	
16	0	0
20	0.04	0.04
50	95.47	95.51
80	2.84	98.35
100	0.79	99.14
200	0.81	99.95
pan	0.05	100.00

Characteristics of Concrete

A number of characteristics of the concrete were investigated as indicated in Table 1. These characteristics are discussed in the following sections. Unless otherwise stated the test results represent the average performance of three specimens from each of three batches of concrete.

Mixture Properties

The initial mixture had a cement content of 635 lb./yd.^{3} (377 kg/m^3) of concrete, a water to cement ratio of 0.47 by weight, a 2 in. (51 mm) slump, an entrapped air content of 3.0% by volume, and a unit weight of 142 lb./ft.^{3} (2275 kg/m^3). A coarse aggregate volume of 60.0% per unit volume of concrete was used. The wax beads constituted 7.8% of the mix by volume, or 114 lb./yd.^{3} (68 kg/m^3), and were accommodated by removing an equal volume of fine aggregate.

A second series of batches was made incorporating 4% to 6% entrained air. The properties of this mixture were not changed significantly from those of the previous one, with the exception that greatly improved freeze and thaw characteristics were observed. An additional series of batches was cast with no entrained air and with no wax to observe the relative performance of such a concrete.

In the field a greater slump was required for effective placement with the available equipment so the water to cement ratio of the first batch was increased to 0.51, which was significantly below the maximum value of 0.55 for internally sealed concrete suggested as a limit by the FHWA. A batch consisted of 6 yd.^{3} (4.6 m^3) of concrete and was mixed and delivered by ready-mix truck from a central plant. For the remaining five batches of field concrete the cement content was increased to 752 lb./yd.^{3} (446 kg/m^3) of concrete and a maximum water to cement ratio of 0.47 was observed. For this concrete, the slump was 3.5 inches (89 mm) and the entrained air content was from 4.0% to 6.0% by volume.

Heat Treatment — Laboratory

For the heat treatment in the laboratory, the concrete specimens were heated in an oven during the initial phases of the study. Subsequently, a small metal heating blanket was obtained. This blanket was similar to the blankets proposed for use in the field and both are described in the FHWA report.(1) The small laboratory blanket was used to heat prismatic and overlay specimens, and a heating chamber was used to heat all cylindrical specimens.

The maximum surface temperature recorded for the small laboratory blanket was only 230°F (110°C), however, and this was not comparable to the field situation where maximum surface temperatures exceeding 300°F (150°) were recorded. Nevertheless, it was shown that the slow blanket heating in the laboratory resulted in no differences in the properties of the internally sealed concrete from those obtained with the oven heat treatment. The similarity of results from the blanket and oven heating are believed to result from the fact that both treatments used a slow rate of heating. Typical heating rates are shown in Figures 2 and 3.

Distribution of Wax Beads

The dispersion of wax beads was checked in the unheated, hardened concrete by microscopic examination. The beads appeared to be uniformly distributed with no concentrations or areas lacking beads.

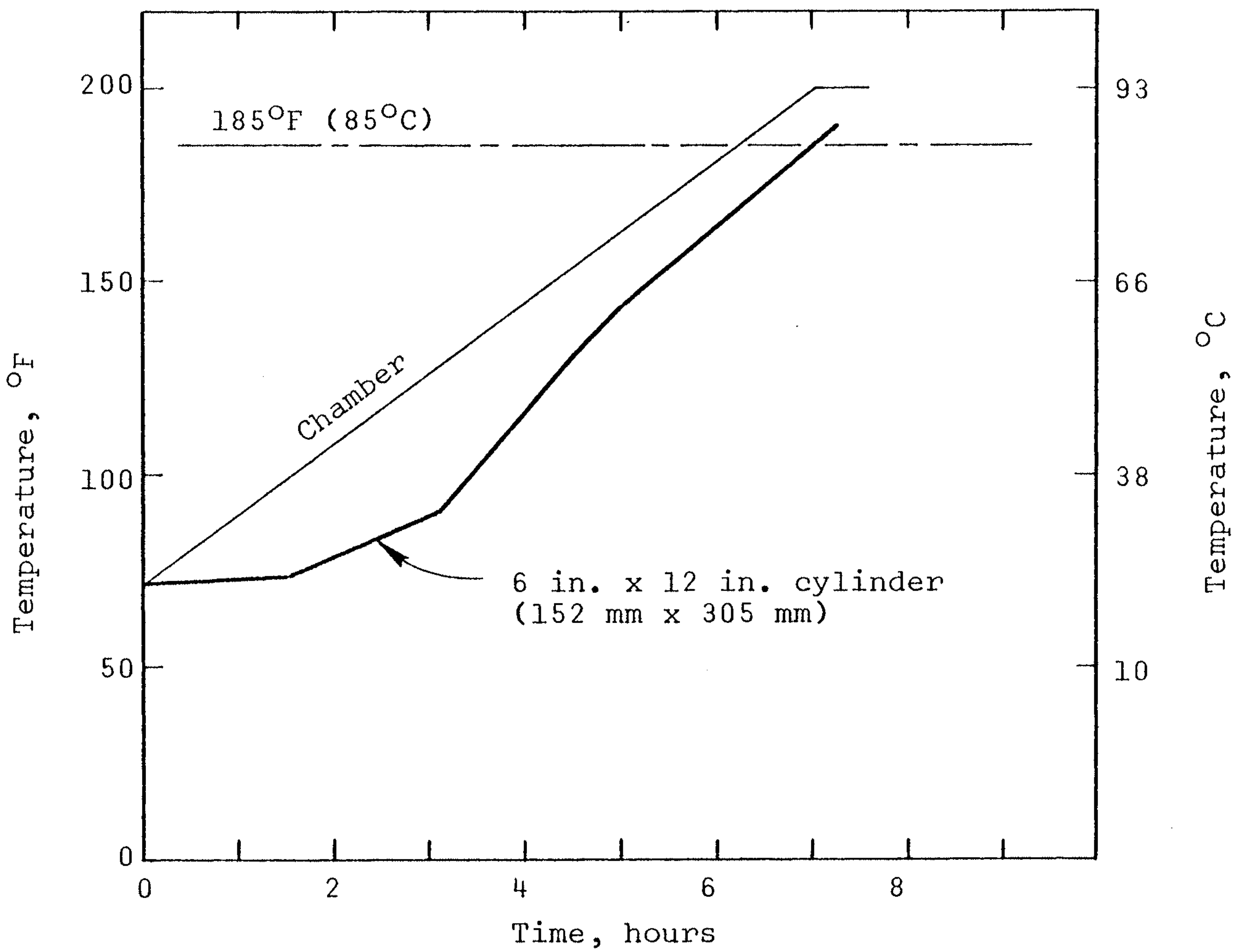

Figure 2. Heating rate at center of cylindrical specimen using heat treatment.

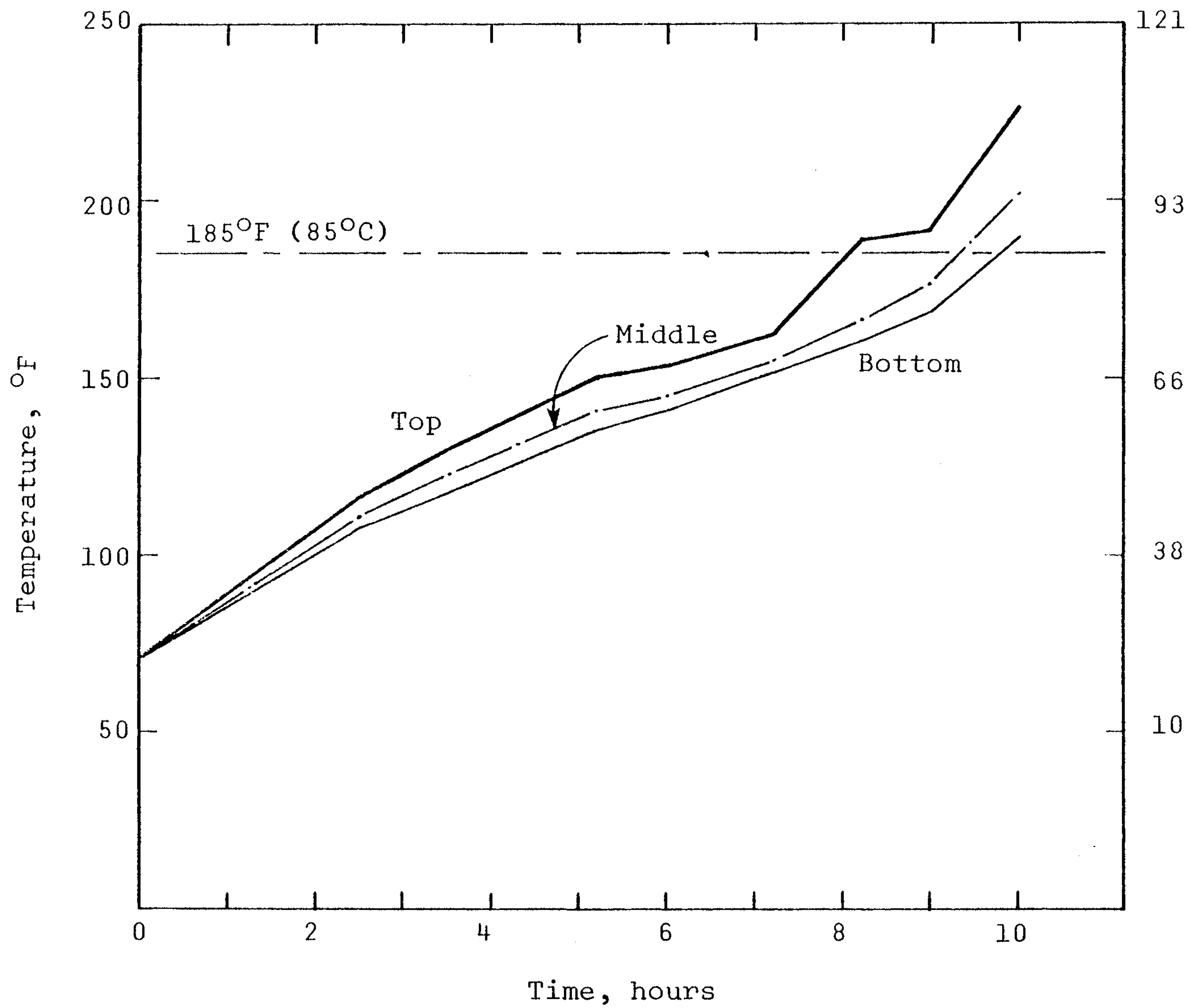

Figure 3. Heating rate at top, middle, and bottom of 3 in. (76 mm) prismatic specimen using electric blanket.

The wax content in the hardened concrete was determined by the linear traverse (Rosiwal) method. This type of examination was performed for cut and polished sections from both unheated and heated concrete specimens to determine the total wax content before heating and the residual wax content after heating. This procedure provided a measure of the amount of wax that had been forced from the initial bead locations into the surrounding paste. The results for a concrete containing limestone coarse aggregate and for one containing a gravel coarse aggregate are shown in Figure 4. Both concretes had additional voids after heating of approximately 6.0% by volume. Since the total wax addition to the concrete was 7.8%, it can be reasoned that 77.0% of the wax flowed into and sealed the paste regions surrounding the original wax bead locations.

This procedure is believed to be reasonably accurate since examination of polished specimens did not reveal any change in the wax contents unless prolonged polishing procedures were used, in which case it is believed some wax was disrupted from the bead locations.

Freezing and Thawing Resistance

The resistance of concrete to the effects of freezing and thawing in water is considered an important characteristic in predicting its long-term performance in field applications. While no test is necessarily equivalent to a specific number of years in service, it has been found that acceptable concretes will meet certain minimum test criteria. In Virginia the test used is ASTM C666, Procedure A, modified to include 2% NaCl in the water surrounding the specimen. This test is considered somewhat severe; however, this severity is offset somewhat by the fact that the test is also modified to allow a 7-day drying period, following 14 days of moist curing, prior to the beginning of the freezing and thawing period. The resulting moisture condition in the concrete test specimen is therefore less severe than when no drying is permitted.

The FHWA research included extensive deicer scaling tests of internally sealed concrete without entrained air in accordance with ASTM C672. The internally sealed concrete performed satisfactorily in this test so this procedure was not repeated in this study. The resistance to freezing and thawing in water (with 2% NaCl) of the non-air entrained concrete was determined in this study and was found not to be acceptable. The mixture was modified to include from 4% to 6% entrained air, and this concrete performed satisfactorily in the test. The resistances to freezing and thawing of the non-air entrained and air entrained internally sealed concretes are shown in Figure 5 along with the resistance of conventional non-air entrained concrete. The acceptance criteria are indicated in Figure 5.

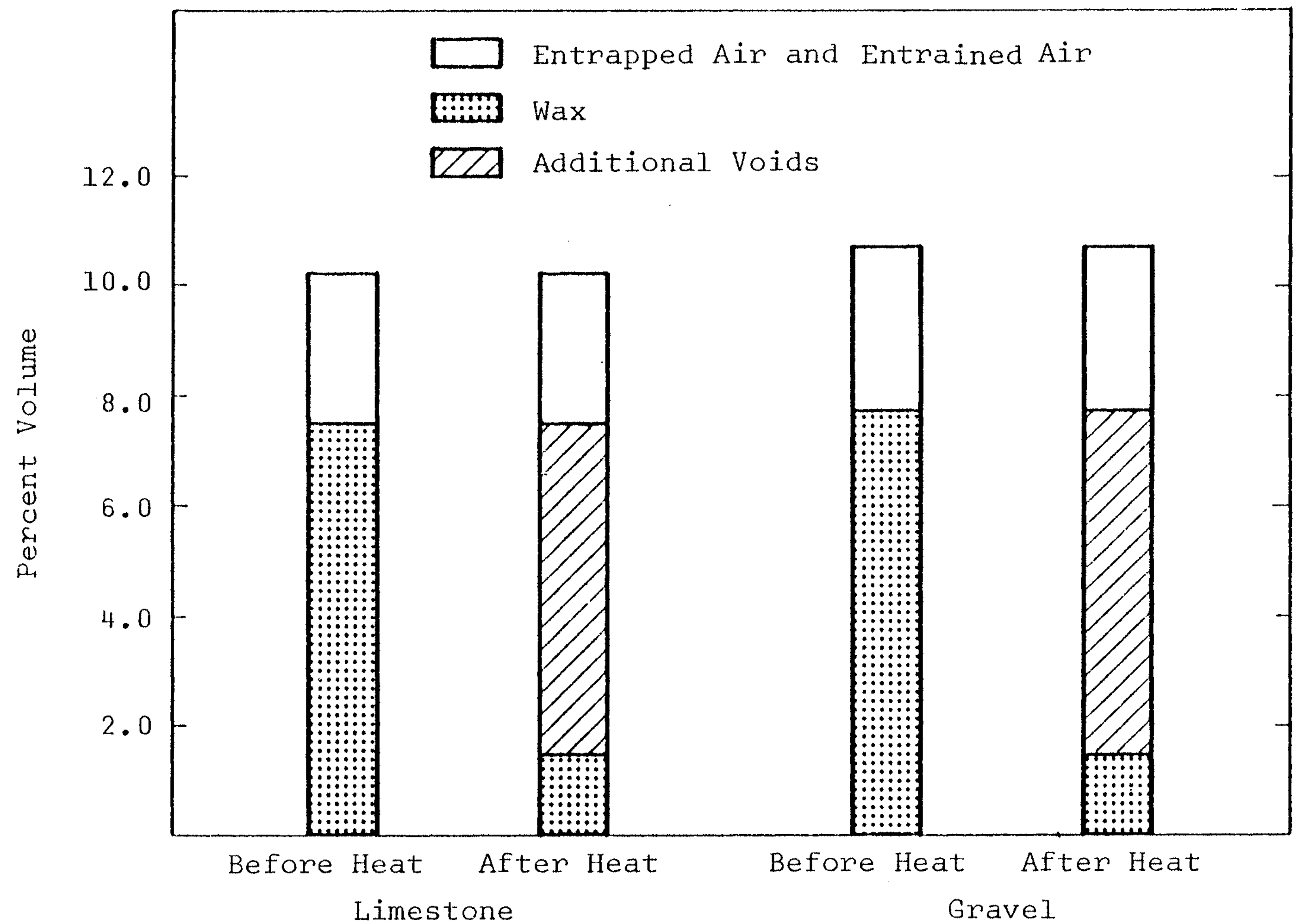

Figure 4. Wax and void contents in hardened concretes before and after oven heat treatment.

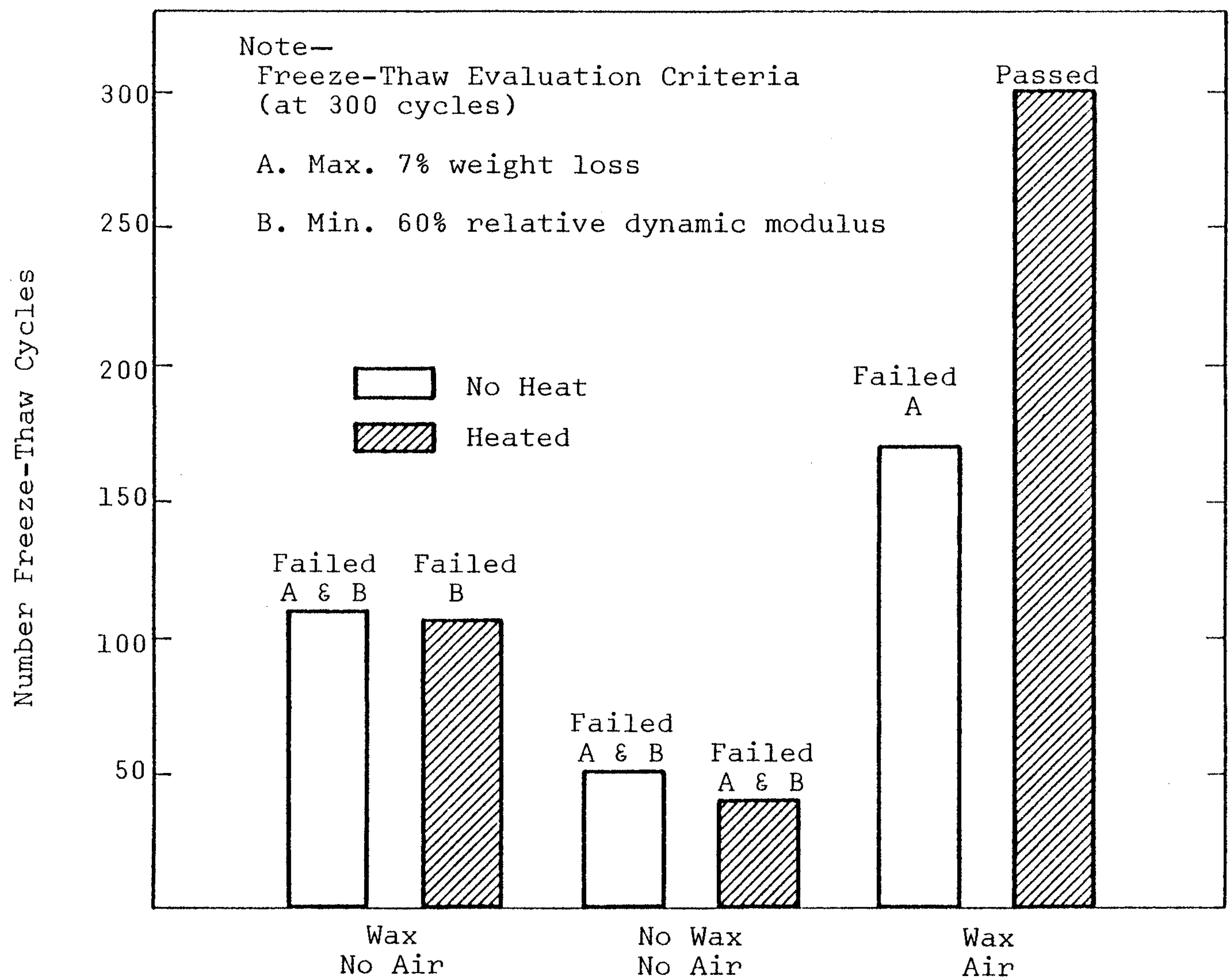

Figure 5. Freezing and thawing resistance of concretes.

It should be noted that the same good performance in the freezing and thawing test was obtained for the air entrained internally sealed concrete heated in the oven and that heated with the electric blanket.

The mode of failure after 54 cycles of the concrete containing wax but not having been heat treated can be seen in Figure 6 to be an excessive weight loss due to surface scaling. The internally sealed concrete with no air entrainment failed by rupturing, as shown in Figure 7, after 54 cycles, even though the scaling resistance was very good. The air entrained internally sealed concrete was durable, both in terms of scaling resistance and resistance to rupturing, for the full 300 cycles.

Compressive Strength

The strength characteristics of the concretes in this study are shown in Table 5. The strengths were determined after an age of 28 days with continuous moist curing; after 21 days of moist curing and 7 days of air drying; and after 21 days of moist curing, 7 days of air drying, and an oven heat treatment for internal sealing.

There was a strength loss of 14.0% from the moist to the heated condition for the non-air entrained internally sealed concrete. At least part of this loss can be explained as a normal occurrence when concrete is heated since, as is apparent from Table 5, the non-air entrained conventional concrete and the air entrained internally sealed concrete experienced similar strength losses of about 7.0% and 5.0%, respectively.

There is apparently no problem with strength in the internally sealed concrete, since all of the specimen groups had average strengths above the desired 4,000 psi (27.6 MPa) level.

Bond Strength

The bond strength between a base layer concrete and internally sealed overlays was determined with overlay shear specimens. The overlays were placed on concrete that had been moist cured for 28 days. Just prior to placement of the overlay concrete the base concretes were cleaned with a wire brush to remove laitance. The overlays were bonded to plain base blocks and to blocks coated with a thin layer of cement paste. Two overlay concretes were used; one containing crushed limestone coarse aggregate and the other a gravel coarse aggregate.

Figure 6. Freeze-thaw specimen with scaling type failure.

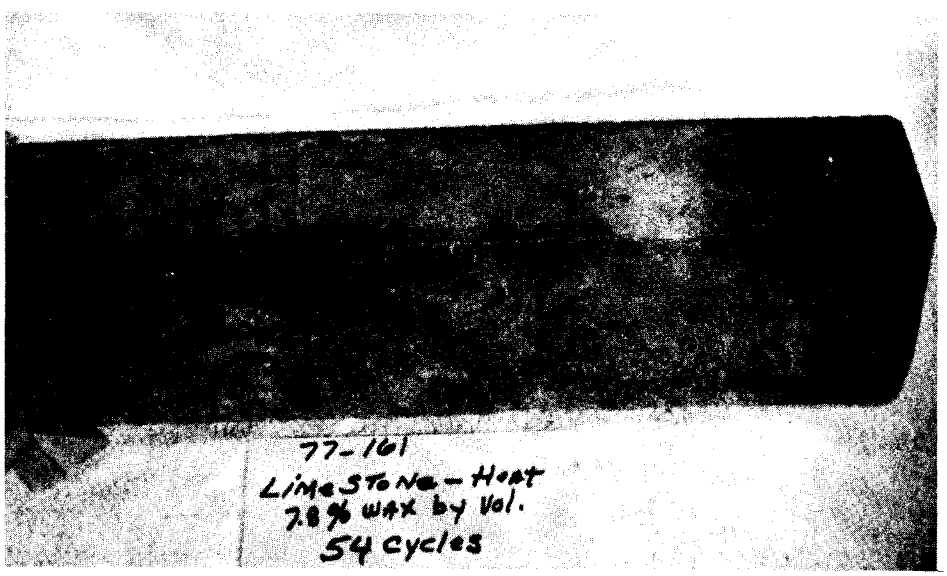

Figure 7. Freeze-thaw specimen with rupture type failure.

Table 5

Compressive Strength of Laboratory Specimens

Cure	Batch	Compressive Strength, psi		
		Wax	No Wax	Wax & Air
28 day moist	1	4,740 4,710	6,830 6,750	4,340 4,770
	2	4,690 4,630	6,520 6,640	4,010 4,640
	3	4,960 4,890	6,600 6,630	4,150 4,690
	Avg.	4,770	6,660	4,430
21 day moist + 7 day air dry	1	4,910 —	6,900 6,900	4,390 4,510
	2	4,850 —	6,940 6,940	4,340 4,340
	3	5,250 —	6,830 6,690	4,600 4,510
	Avg.	5,000	6,930	4,450
21 day moist + 7 day air dry + heat treat	1	3,630 3,620 3,780	6,070 — —	4,350 4,280 —
	2	4,550 4,580 4,390	6,320 — —	4,290 4,010 —
	3	3,960 3,860 4,090	6,100 — —	4,200 4,160 —
	Avg.	4,050	6,160	4,220

Note: psi x 0.006895 = MPa.

The average shear strengths for the several overlay conditions are shown in Figure 8. There it may be observed that the average strength for each condition is within or above the 200 to 400 psi (1.4 to 2.8 MPa) range indicating good to excellent bond strength. With good vibration it is apparent that adequate bond strengths can be obtained with and without the use of a bonding agent.

Effectiveness of Sealing

The effectiveness of the internal sealing was determined in several ways. As discussed earlier in the section on the distribution of the wax, approximately 77% of the wax was found to have flowed from the original wax bead locations into the surrounding paste. This finding can be taken as an indirect measure of the sealing effectiveness of the wax.

Another procedure that is relatively easy to perform involves the absorption of water into the concrete. With this procedure, there is an obvious difference in the heated and unheated specimens containing wax, as shown in Figure 9 where the amount of absorbed water in the unheated specimens is seen to exceed that in the heat treated specimens by a factor of seven. In this test only a mild drying temperature of 120°F (50°C) was used so that neither initial nor additional melting of the wax would occur.

A third test that is more involved and more convincing with respect to the sealing effectiveness of the internally sealed concrete was the soaking of concrete specimens in an NaCl solution. In this test, sealed and unsealed concrete cylinders containing wax were soaked for 90 days in a 2% by weight NaCl and water solution. The amounts of chloride ions penetrating to the 0.5 in. (13 mm) and 1.25 in. (32 mm) depths were determined by titration analysis. As shown in Figure 10, the chloride concentration of the greater depth did not exceed the original or baseline chloride content of the concrete after 90 days in either the heated or unheated specimens. It is significant, however, that at the shallower depth a large amount of chloride exceeding the corrosion threshold level was found in the unheated specimens, while practically no chloride was found to have intruded the internally sealed concrete. This test provides definite proof of the effectiveness of the internally sealed concrete in preventing intrusion of corrosive substances.

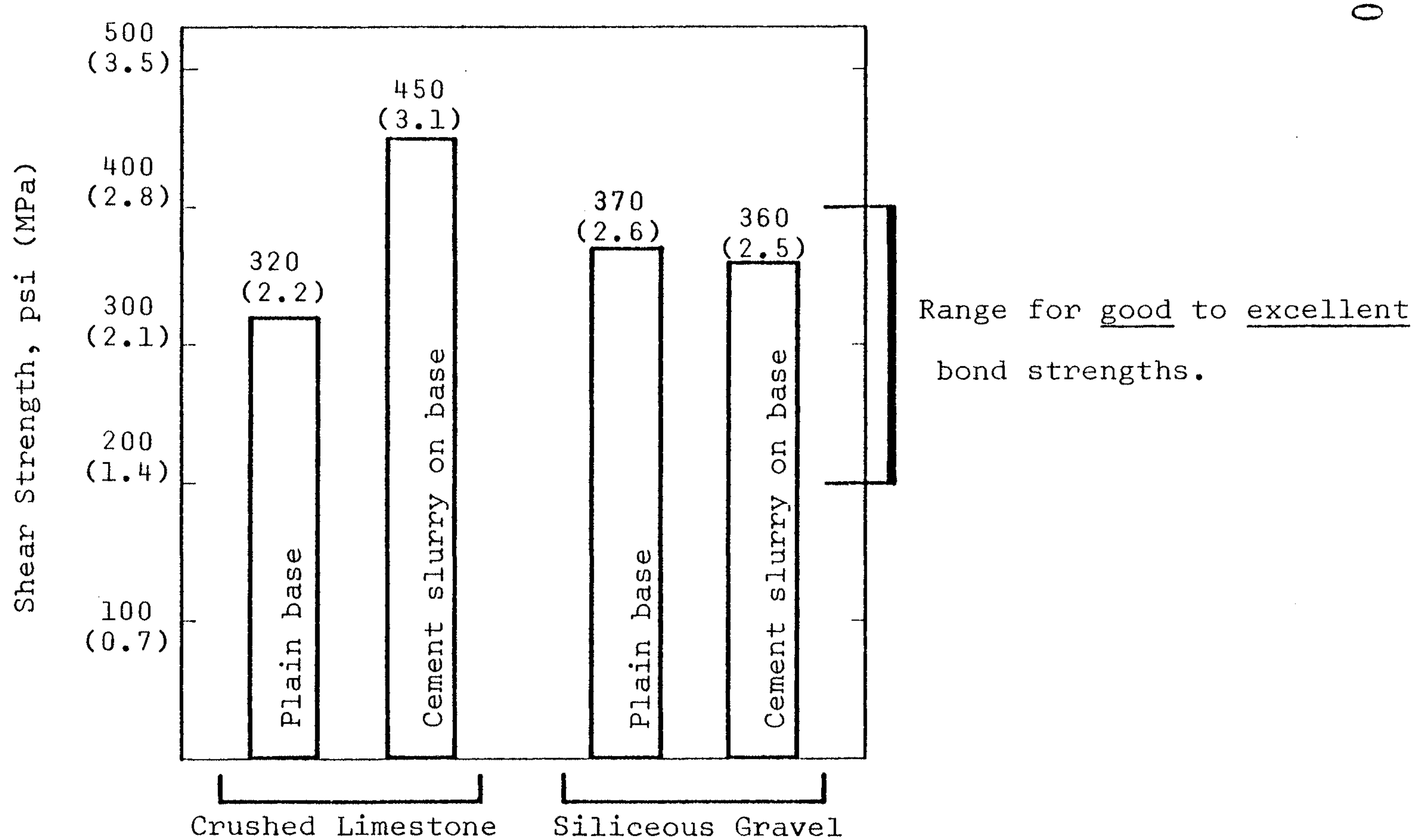

Figure 8. Shear strengths developed between base layer concretes and internally sealed overlays.

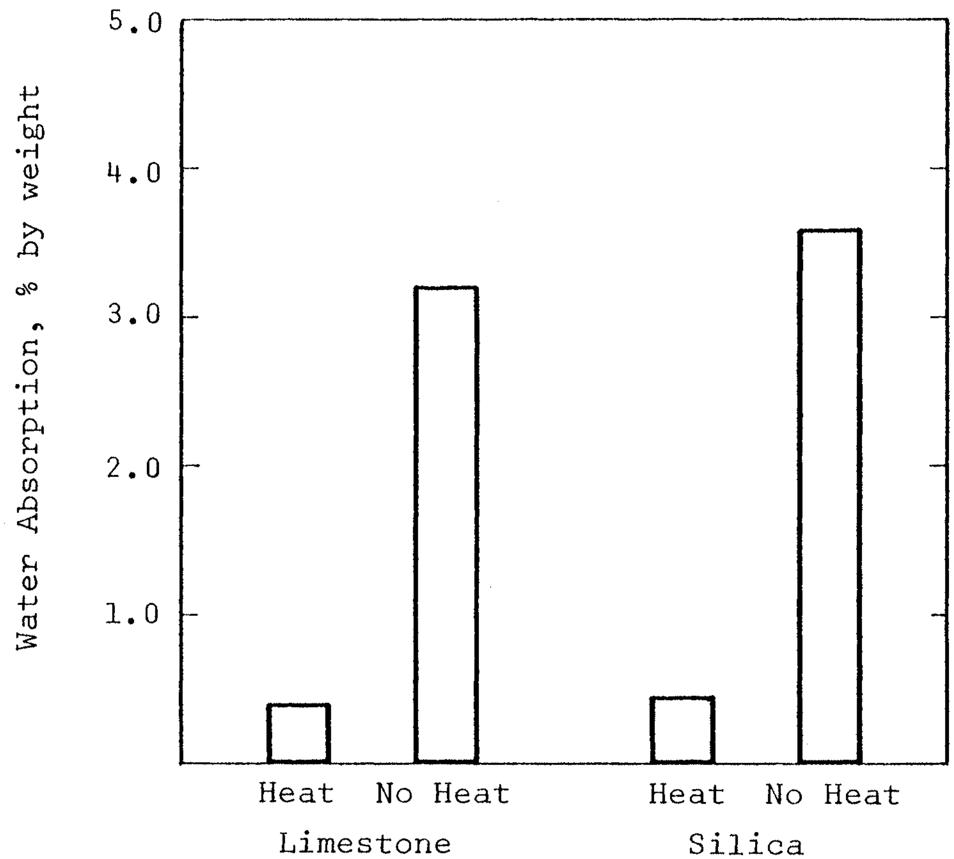

Figure 9. Absorption quantities for wax concretes with and without the oven heat treatment.

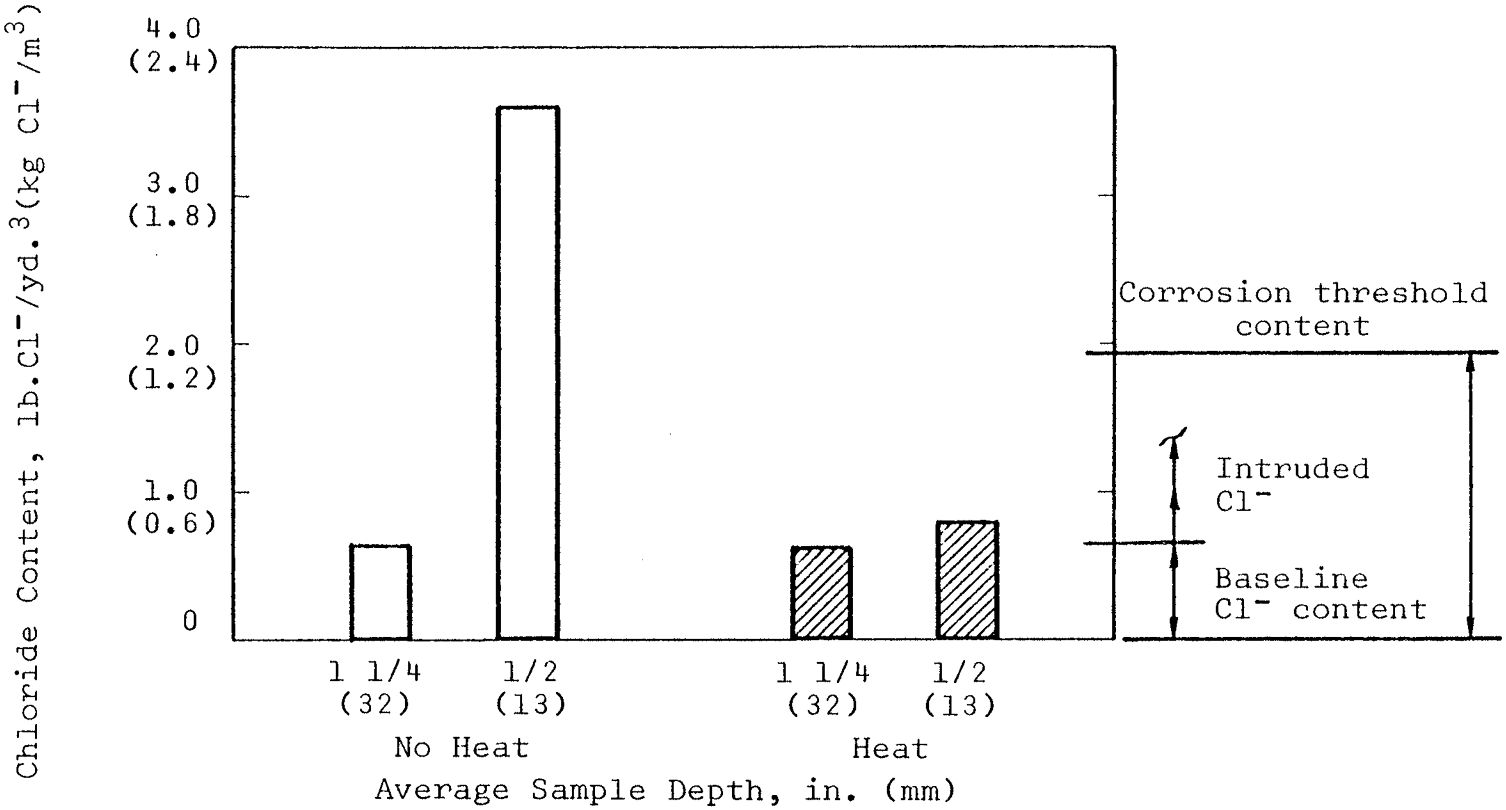

Figure 10. Chloride ion contents in 3 in. x 6 in. (76 mm x 152 mm) concrete cylinders after 90-day soak in water with 2% NaCl by weight.

Cracking of Laboratory Specimens

Some cracking was observed in laboratory concrete specimens heated in the oven. This cracking was not visible to the unaided eye and was identifiable only on polished specimens with the use of a microscope. The cracking was found to extend only about 0.5 in. (13 mm) from the peripheral area into the specimens, and was found to occur primarily with internally sealed concrete containing gravel coarse aggregate. Less severe cracking occurred with similar concrete containing a crushed limestone coarse aggregate.

A possible explanation of this phenomenon is found in Figure 11,[2] where the thermal coefficients of expansion of mortars and concretes containing crushed limestone are shown to be considerably lower than those of concretes containing gravel. The greater expansion of the gravel concrete may account for the appearance of pronounced cracking and may suggest a conservative approach of not using gravel coarse aggregate in internally sealed concrete.

Figure 12 shows polished sections of the concrete containing wax and silica gravel coarse aggregate. The section on the left was unheated and the section on the right was heat treated. The cracks in the heated section have been marked with dark ink to indicate their locations.

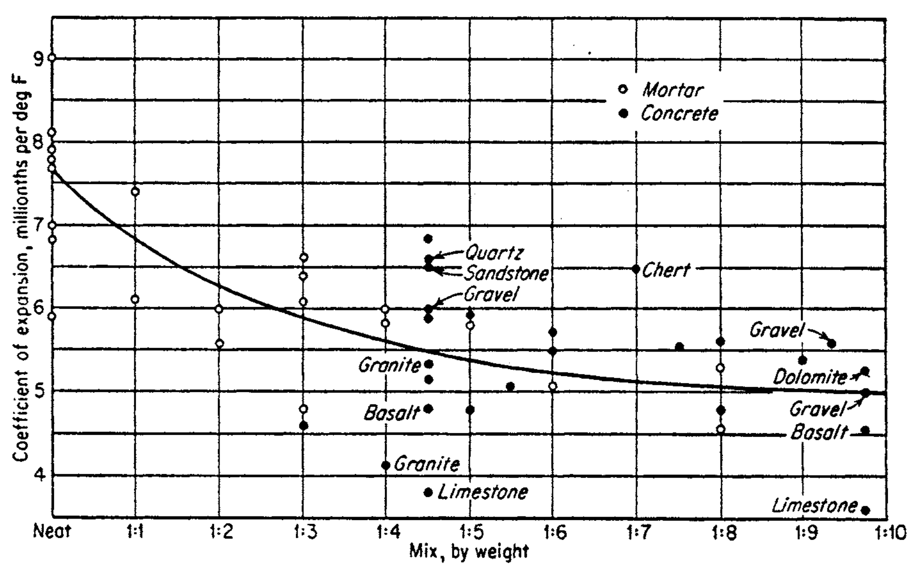

Figure 11. Thermal coefficients of expansion of neat cements, mortars, and concretes.[2]

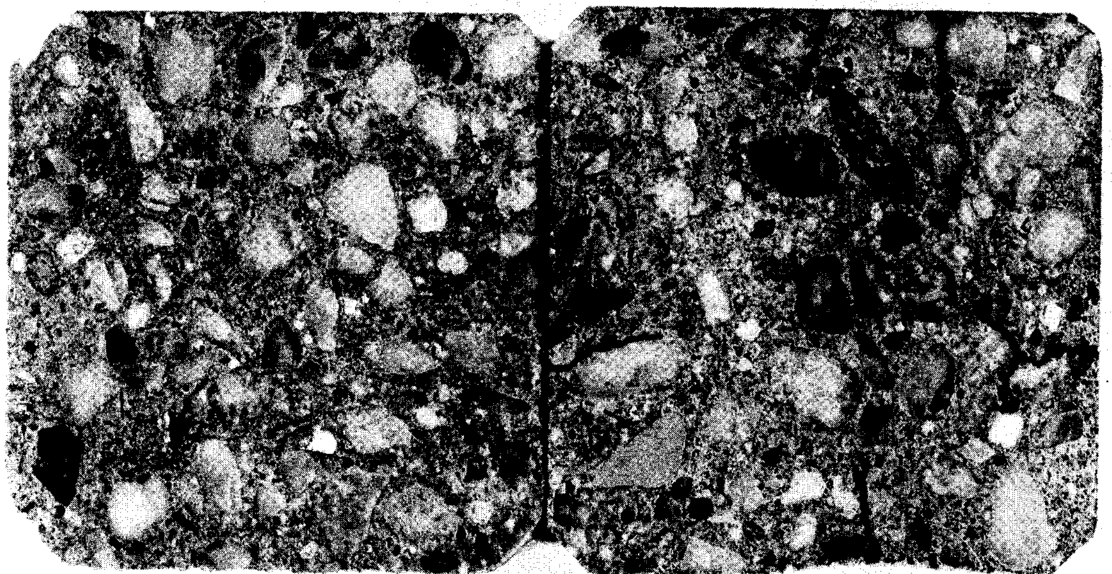

Figure 12. Polished sections from unheated (left) and heated (right) 3 x 3 x 11 in. (76 x 76 x 279 mm) prisms.

Field Testing

The field portion of the study was intended primarily to evaluate the heat treatment method used for internal sealing of the overlay concrete. The bridge decks were constructed using two-course construction with the protective concrete being placed in only the upper 2 in. (51 mm) of the decks. Because the cement factor of the overlay concretes had to be increased from 635 lb./yd.^{3} (377 kg/m^3) of concrete, which was used in the laboratory phase of the study, to 752 lb./yd.^{3} (446 kg/m^3) of concrete for effective placement in the field, the characteristics of the field concrete were also determined to verify that they were similar to those determined for the laboratory specimens.

Specimens of the field concrete were prepared and tested for wax distribution, compressive strength, water absorption, and freezing and thawing resistance. Values for these critical characteristics were essentially identical to those measured for the laboratory specimens. Since the field data were basically the same as the laboratory data many of the field results are not shown.

The total evaluation of the field operations included observations made both before and after the heat treatment. Therefore, the discussion in the following sections is directed in chronological order to the overlay placement, the field heat treatment, and the thermal cracking of the overlay concrete.

Placement of Overlay

The protective overlay, or stage two, portion of the two-course spans was placed using the transverse screed shown in Figure 13. This screed was satisfactory for consolidating concrete with a slump in the 2 in.-to 4-in. (51-to 102-mm) range. The oscillating components of the screed can be viewed from left to right in Figure 13 and consisted of a suspended pneumatic vibrator, a rotating auger and metal roller, and a metal float. The special provision for this project is in the Appendix.

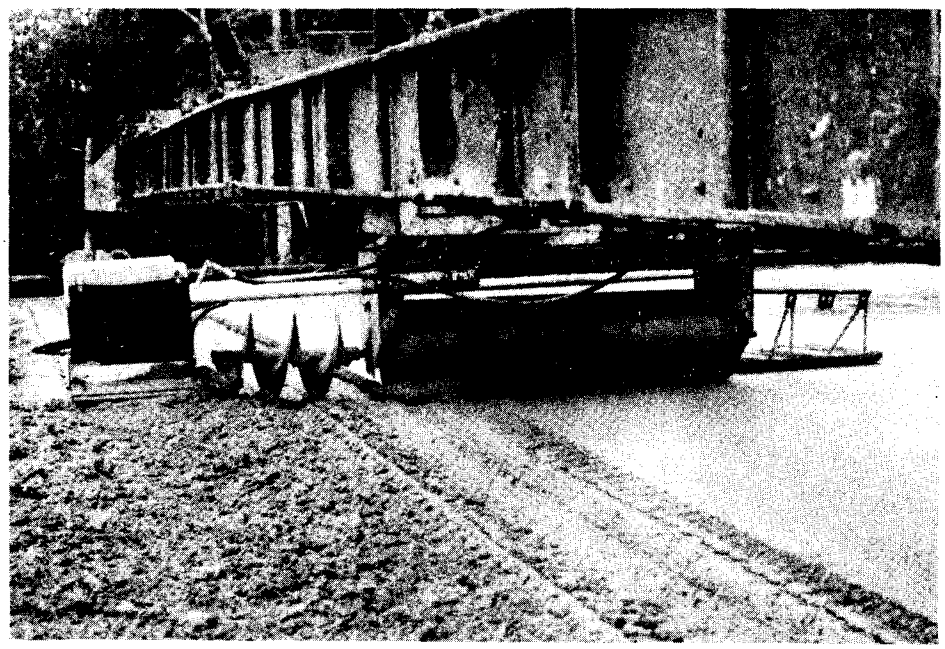

Figure 13. Transverse screed used in placement of overlay concrete.

Placement Data

Concrete for the overlays was delivered to the job site in ready-mix trucks. Six truckloads of 6 yd.^{3} (4.6 m^3) each were required. At the time of delivery the air temperature and relative humidity were recorded along with the temperature, slump, air content, and unit weight of the concrete. These data are presented in Table 6. The air temperatures and relative humidities indicate good conditions for placement of concrete. There was little or no wind during the placement.

Table 6

Overlay Placement Data

Ready-Mix Truck Time	Air Temp., °F[a]	Relative Humidity, %	Concrete Temp., °F[a]	Slump in.[b]	Air Content % Vol.	Unit Weight lb./ft.^{3}[c]
#1/ 9:15 a.m.	65	80	78	3.0	3.5	135.4
#2/ 10:55 a.m.	70	78	74	3.0	4.5	135.9
#3/ 11:55 a.m.	72	76	76	4.3	5.5	133.8
#4/ 12:17 p.m.	70	80	77	3.3	4.5	135.5
#5/ 2:57 p.m.	72	84	78	3.8	4.5	134.8
#6 4:00 p.m.	74	77	78	4.0	6.0	134.8

a (°F - 32) x 5/9 = °C

b in. x 25.4 = mm

c lb./ft.^{3} x 16.02 = kg/m^3

The concrete data in Table 6 show uniformly good control for the six batches of concrete. The compressive strengths of cylinders from these batches before and after heat treatment are shown in Table 7. The average strength of the heated specimens was reduced by only 5%. This result is compatible with the findings presented earlier for the laboratory batches.

Bond Interface

The surface of the base layer, or stage one, concrete on two of the three spans was roughened by raking while the concrete was still fresh to enhance the bond between it and the

Table 7

Compressive Strength of Field Cylinders
Containing Wax and Air

Cure	Batch No.	Compressive Strength, psi
28-day Moist	1	4,110
	2	4,330
	3	3,990
	4	4,330
	5	4,030
	6	4,550
	Avg.	4,220
21-day Moist + 7 day Air Dry + Heat Treatment	1	3,650
	2	4,170
	3	3,860
	4	4,240
	5	3,940
	6	4,330
	Avg.	4,030

Note: psi x 0.006895 = MPa

overlay concrete. An area of the raked finish is shown in Figure 14, where it may be observed that numerous coarse aggregate particles and balls of mortar were pulled from the surface during the raking process. The unbonded aggregates and mortar on the base layer surface had to be manually chipped from the deck surface and blown clear with compressed air prior to placing the overlay concrete.

In order to more fully evaluate the raked finish a comparison was made between it and a screeded finish with no raking as shown in Figure 15. The screeded finish was applied to the third of the three experimental spans. These two types of surfaces were overlaid and subsequently cored.

The core in Figure 16 is from the raked surface. Two zones have been circled along the irregular bond interface where loose aggregate and poor consolidation of the concrete into the raked valleys resulted in a weakened bond. The core in Figure 17 is from the overlay and stage one surface that was screeded but was not raked. The bond interface in this case is uniformly good with no unsound zones. This screeded base layer is essentially the same as used for laboratory specimens which have produced excellent shear strengths. It is also similar to base layers which have been successfully used in two-course construction in Virginia.[3] (Note: In Figures 16 and 17 shrinkage cracks are outlined but these are not related to the bonding and will be discussed later.)

From the information available, it appears that the raked finish creates several conditions that are detrimental to achieving good bond, and that the screeded finish is all that is needed for good bond. It should be emphasized, however, that in any case if foreign materials or excessive laitance is present on the base layer surface, the deck should be sandblasted prior to placing the overlay concrete.

The fresh concrete overlays were placed on the base layer concrete without the application of a bonding material. It is assumed that with adequate preparation of the base layer and with proper consolidation, the strengths of the plain bonded overlays should be similar to the strengths indicated earlier in Figure 8 for laboratory specimens. The use of a plain bond interface between fresh and hardened concretes has been indicated in previous studies in Virginia.[4,5]

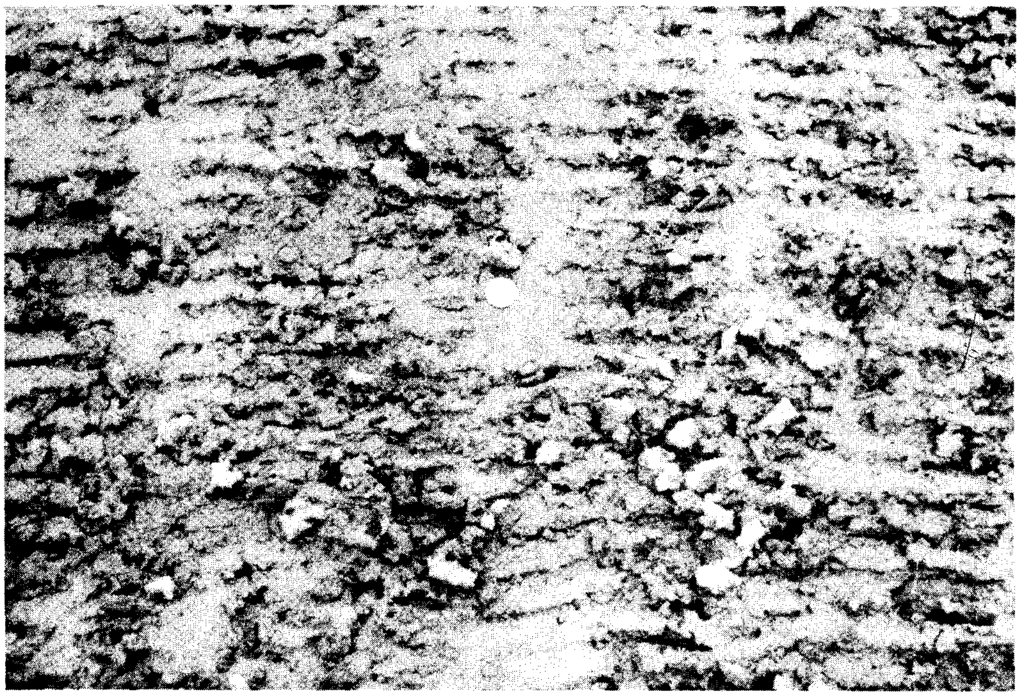

Figure 14. Raked surface of stage one concrete with quarter dollar shown for size.

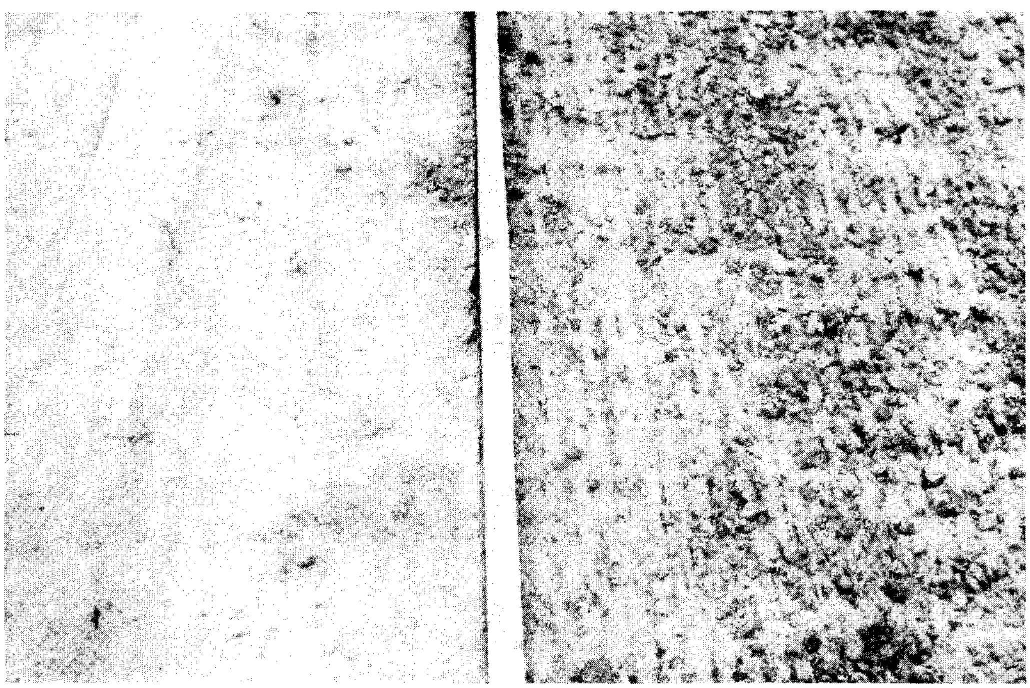

Figure 15. Stage one concretes at expansion joint showing screeded finish (left) and raked finish (right).

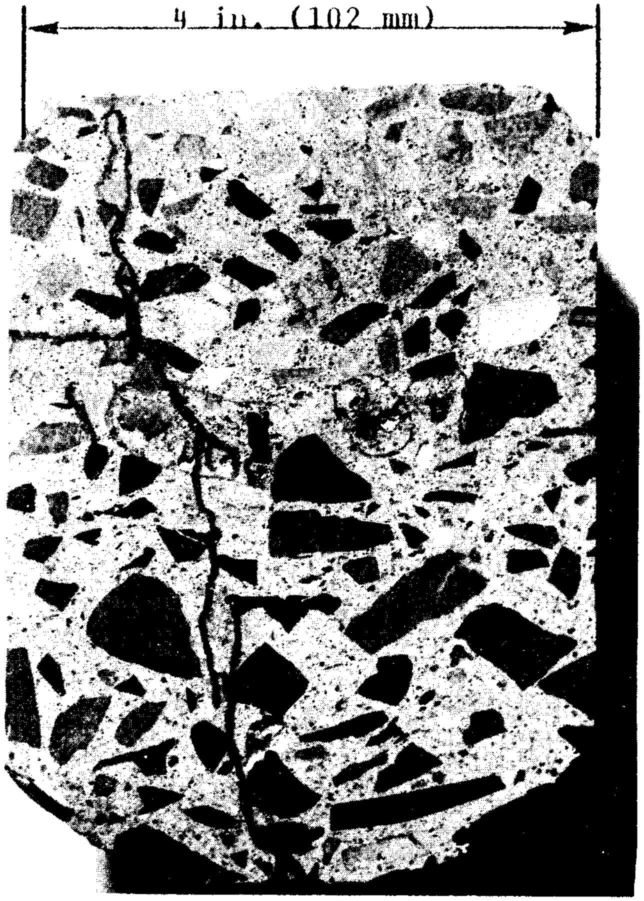

Figure 16. Core through overlay and raked base layer.

Figure 17. Core through overlay and screeded base layer.

Heat Treatment — Field

The heat treatment of the overlay concretes on the bridge structures was accomplished using the electric blanket heating system owned by the FHWA.(6) The blanket system was loaned to the contractor along with a crew of three people from the FHWA for technical assistance. The heating of three spans was completed in six runs over a period of 4 days. The average length of a heat run varied from 7 to 10 hours, with an additional 1.5 hours being required to assemble the system prior to each heating.

The basic elements of the blanket heating system are shown in Figures 18 through 20. In Figure 18 the 550-volt generator is shown in the background with the transformer unit in the foreground. The control panel for the system contained an on-off switch, a voltage indicator, and ammeters for each circuit to the blankets. The control panel is shown in Figure 19. Several of the heating blankets can be seen in Figure 20, along with the dual layers of 3 in. (76 mm) thick insulation and rope netting provided to stabilize the insulation during windy conditions.

The temperature of the concrete was recorded during the heat treatment at several locations using thermocouples. The thermocouples were placed at the surface of the concrete, at the 2-in. (51-mm) depth, at the 2.5-in. (64-mm) depth, and at the 6.5-in. (165-mm) depth. A record of typical temperatures is shown in Figure 21 for one slab with heat applied over approximately one-half of the slab area.

There was a rapid increase in the surface temperature accompanied by uniformly slow increases at the lower depths at which the temperatures were monitored. The temperature changes at various depths are shown in Figure 21. The maximum temperature gradient for the depths indicated was reached approximately 3 hours after the heating began and was maintained throughout the remainder of the heating periods.

A transition in the temperatures between the heated and unheated areas of the slab was provided by a partial heat zone. The heat produced in the end panels of the blankets varied from a maximum at the inside edge to zero at the outside edge so that a 15 in. (0.4 m) longitudinal transition zone was created along the edge of the heated zone. Visual evidence of the transition zone can be seen in Figure 22 where the fully heat treated area appears darker than the area that was under the end panel. A similar transition zone was to be provided in the transverse direction by not covering the edge blankets in the system with insulation, and thereby allowing rapid heat loss to the atmosphere along these strips.

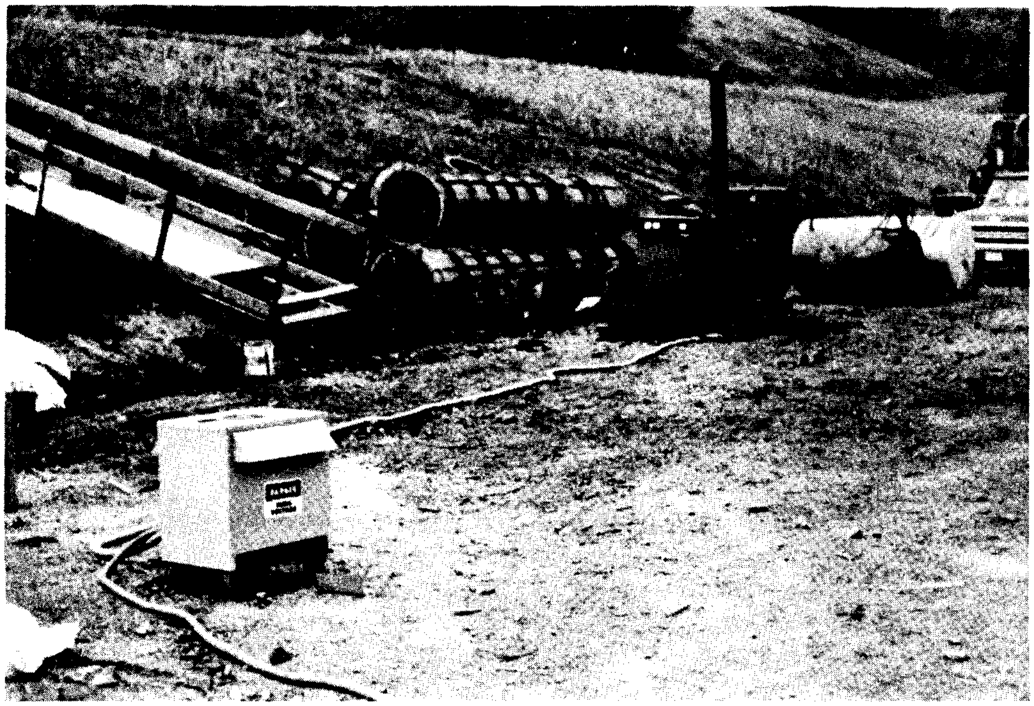

Figure 18. Generator (background) and transformer.

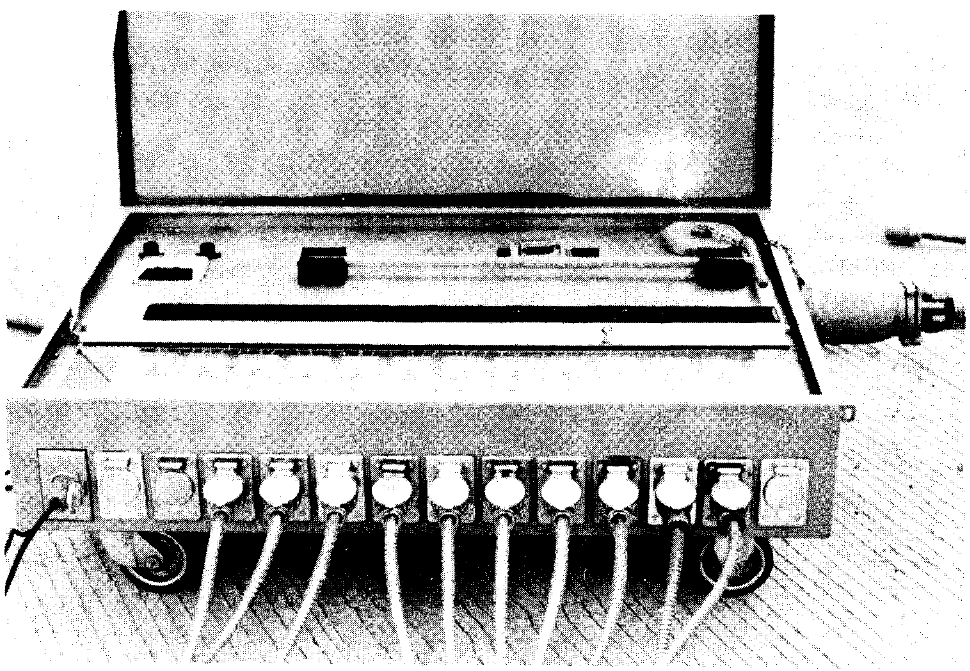

Figure 19. Control panel for electrical distribution to heating blankets.

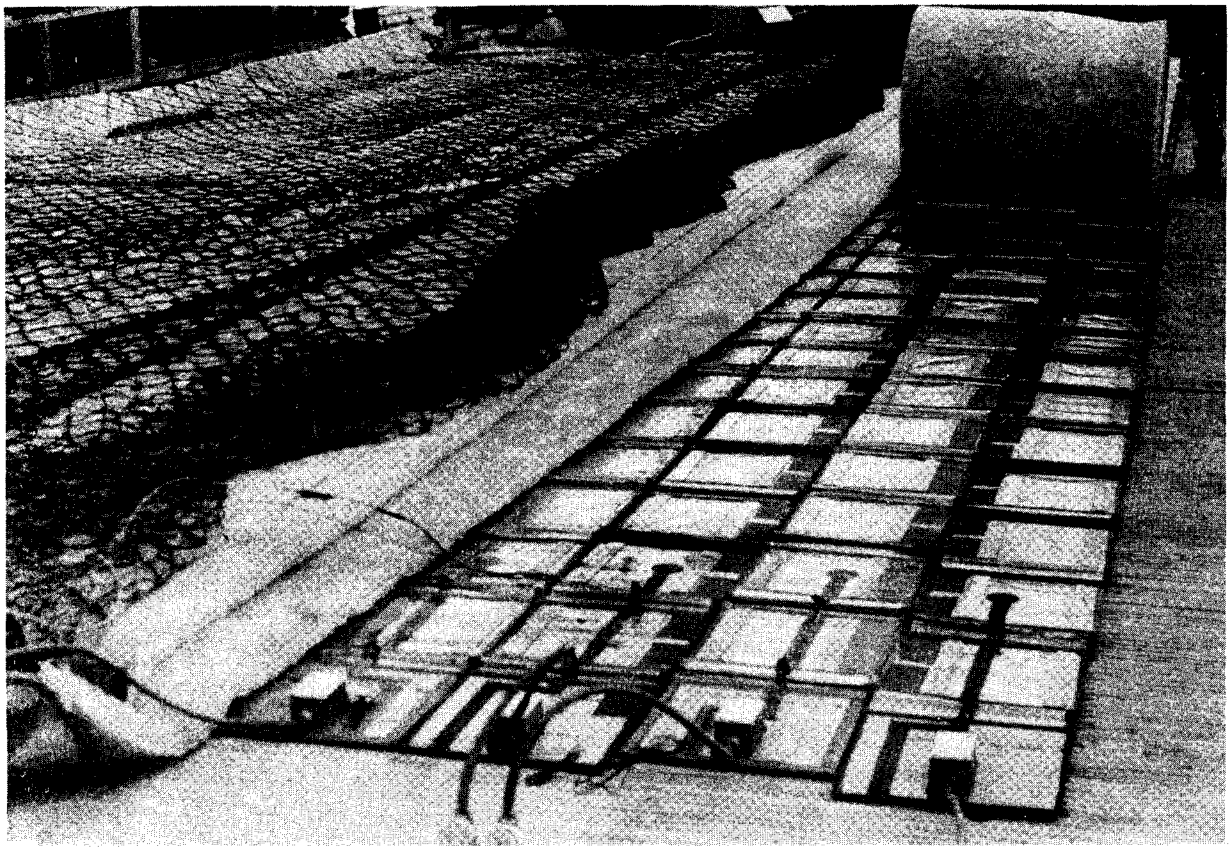

Figure 20. Heating blankets on deck with insulation and netting.

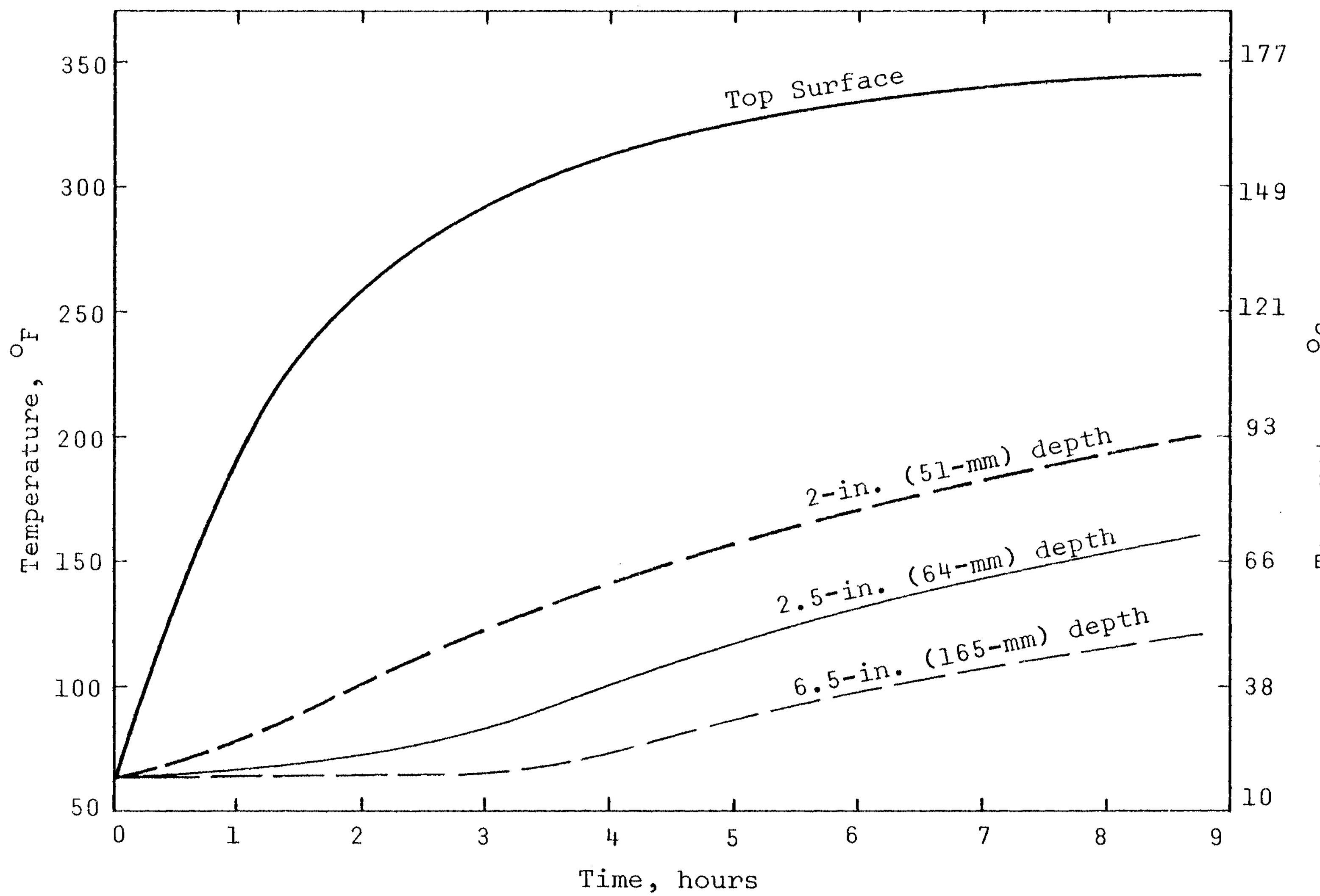

Figure 21. Temperature changes in bridge deck slab.

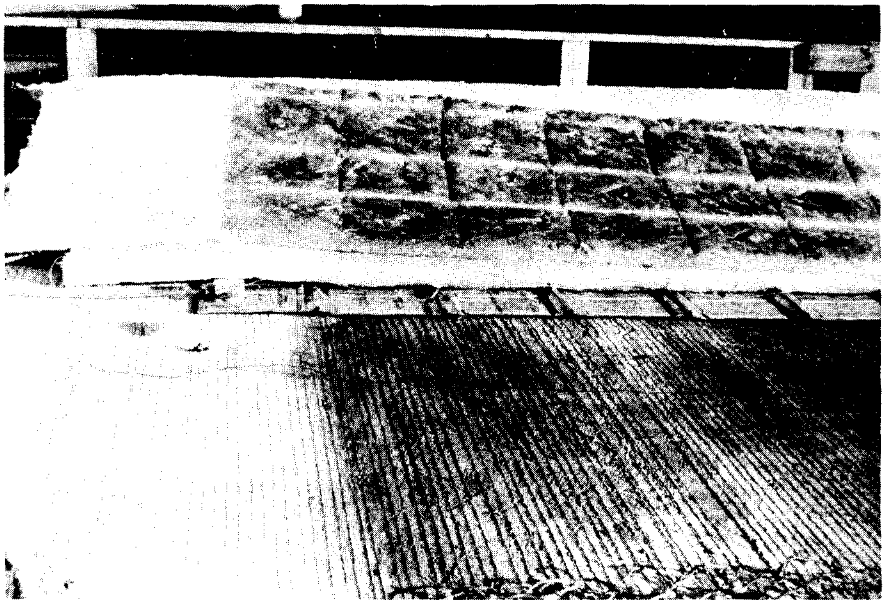

Figure 22. Transition on deck between unheated concrete (left) and darkened, heated concrete (right).

Cracking of Field Concrete

It was found that fine cracks developed during and after the heat treatments. These cracks will be referred to as thermal cracks, even though the precise temperature and restraint conditions for their occurrence cannot be stated. In order to develop an understanding of the thermal cracking it is helpful to study the crack patterns with respect to the locations of the heating blankets on the spans. Figure 23 shows the blanket locations for the six heat runs and the locations of the cracks that were observed on the spans. The overlapping peripheries of the six heated areas are denoted by the lines and shadings and the numbers one through six, which correspond to the sequence of the heat runs. The numbers on the cracks refer to the particular heat run during or after which the cracks were observed.

Several types of cracking were observed and, in general, it can be seen in Figure 23 that each type was associated with the peripheral zone of the blankets. In order to visualize why cracking took place in some areas and not in similar areas, it may be helpful to observe the ambient temperatures that existed during each heat treatment. These are shown in Figure 24.

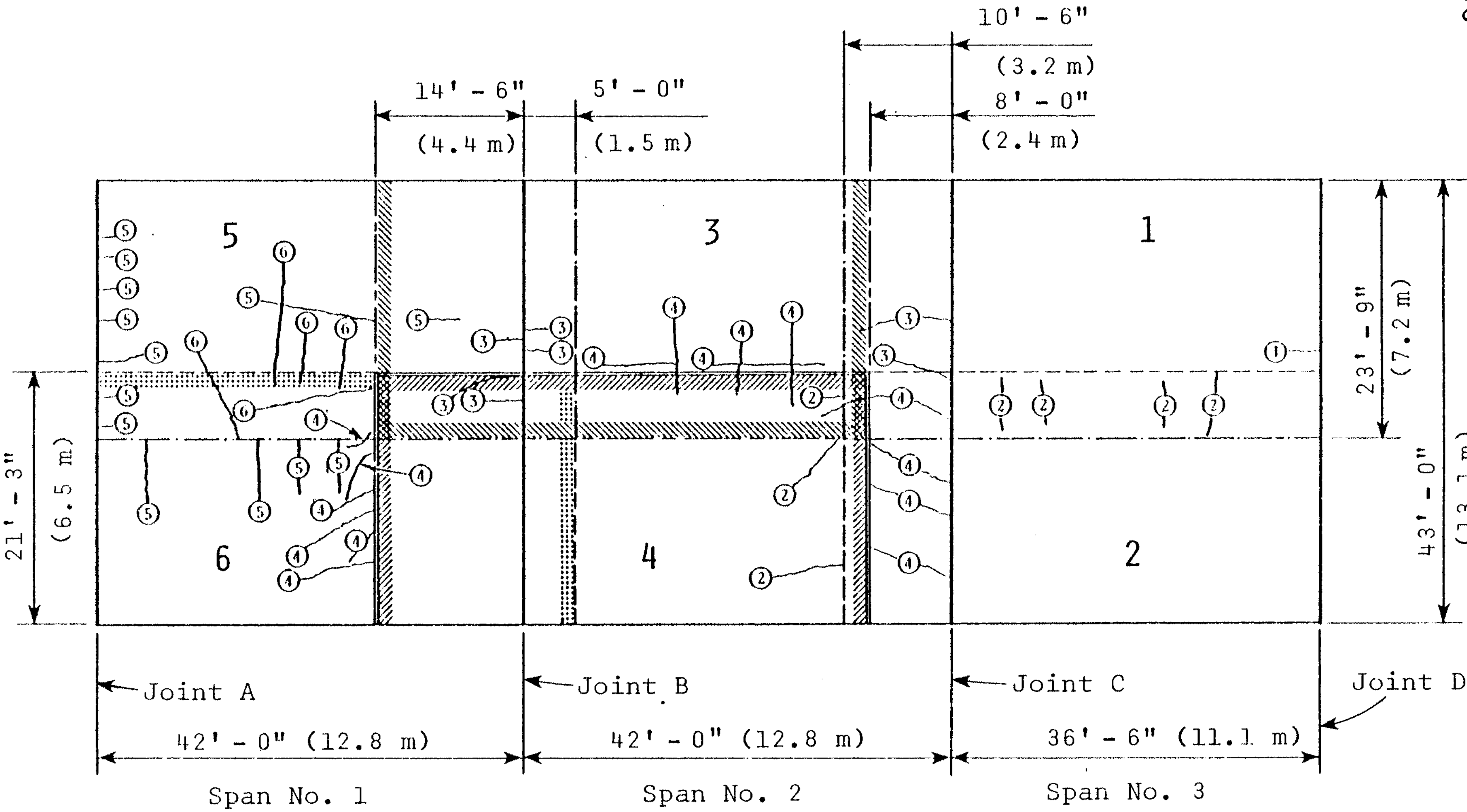

Figure 23. Six heat runs and related cracking.

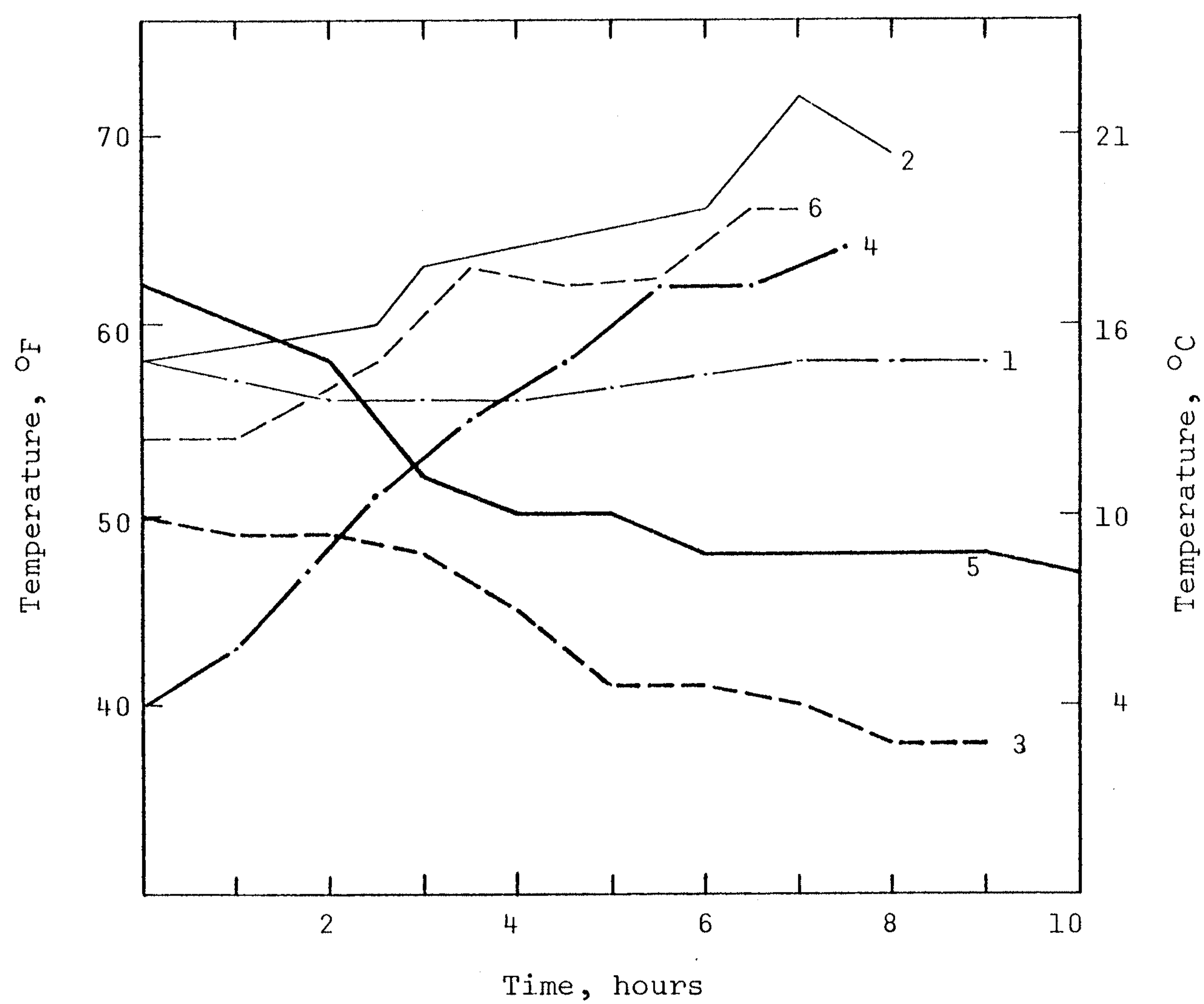

Figure 24. Ambient temperatures during six heat runs.

Some relationships between the crack patterns in Figure 23 and the temperature data in Figure 24 can be noted. First, it can be observed that for the heat runs which ended during cooler ambient temperatures, namely runs number three and number five, short cracks of from 2 to 3 ft. (0.6 to 0.9 m) in length occurred at the expansion joints. These cracks were perpendicular to the joints and pachometer readings showed them to be directly over the longitudinal reinforcing bars. It is probable that the concrete cooled rapidly in these areas due to the exposed joint face and that the cracking, which was observed to take place during the cooling period after the removal of the blankets, was due to internal restraint. This problem might be avoided by forcing strips of insulation into the joints to prevent rapid heat loss during cooling.

Next, it can be inferred from Figure 24 that the unheated concrete would have reached its coldest state during the night hours after heat run number three when the ambient temperatures were the coldest for the 4-day period. This was, in fact, verified by thermocouple readings in the unheated concrete prior to heat run number four. Therefore, the thermal expansion during heat run number four should be expected to have had the greatest tendency to crack the surrounding cool concrete. This can be confirmed by observing the crack patterns surrounding heat area number four in Figure 23. For clarity, the area of heat run number four and the related cracking have been reproduced separately in Figure 25 to a slightly larger scale than in Figure 23. The crack pattern in Figure 25 suggests two actions that might have been used to eliminate or reduce the thermal cracking. First, if the blankets had been used for full heating on only span number two the longitudinal cracking of concrete at the ends of the blankets might have been eliminated, since unheated concrete would not have been acting to restrain the transverse expansion of the heated area. Next, additional blankets without insulation might have been added to the unheated area on span number two adjacent to the heat run to reduce the thermal gradient from the heated to the unheated area. Reduced thermal gradients in the transverse direction might have eliminated or reduced the transverse cracking adjacent to the heated area. The longitudinal crack at the blanket edge near the center of the roadway possibly could have been eliminated by using a wider heat transition zone also.

It may be observed in Figure 25 that no cracking occurred at joint B after heat run number four. Some cracking did take place at joint B after heat run number three, however, as is shown in Figure 23. The difference in cracking can be attributed to the difference in the ambient temperatures at the end of those two heat runs, as depicted in Figure 24. The ambient temperature at the end of heat run number four was considerably higher than that at the end of heat run number three. The slower cooling rate after heat run number four, therefore, can be creditied with preventing the cracking at the joint.

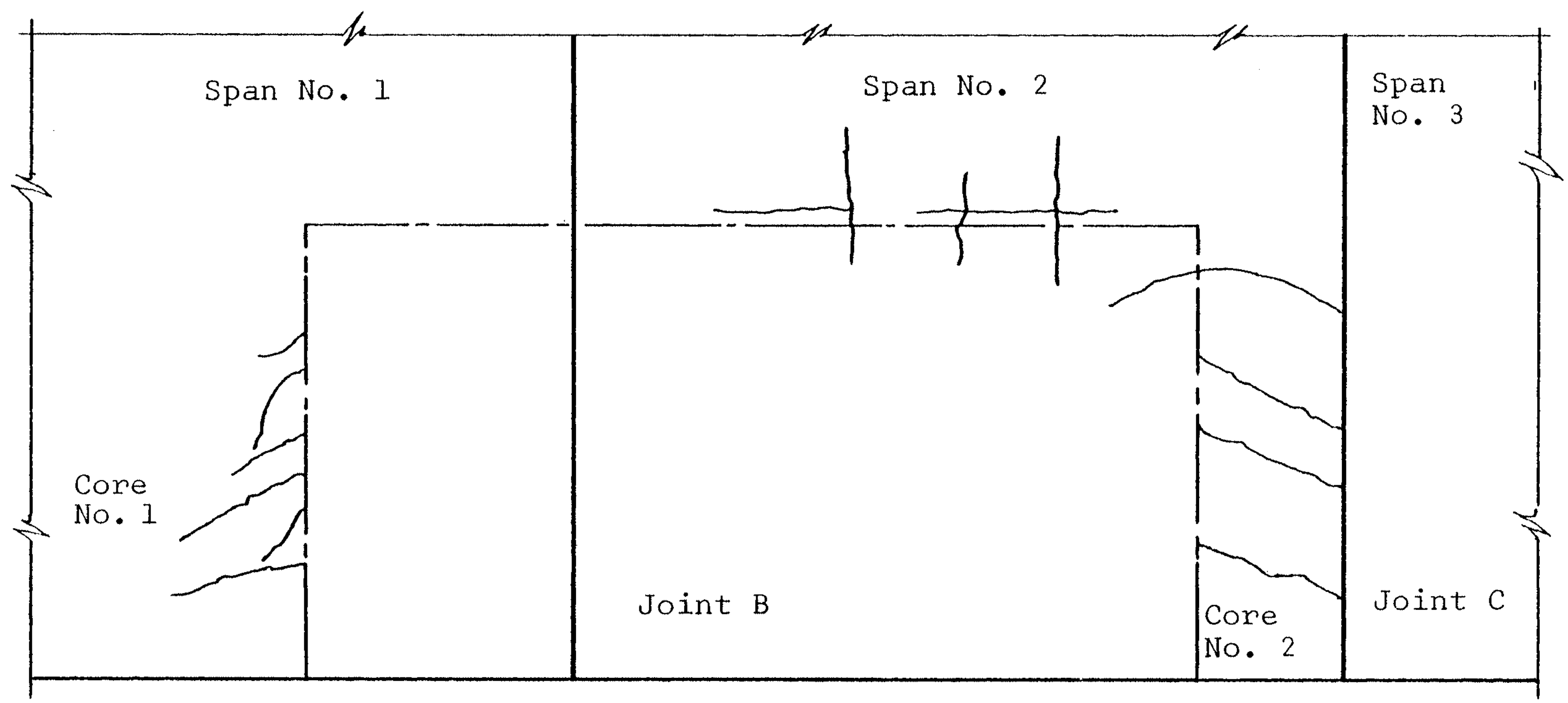

Figure 25. Heat run number four and related cracking.

Similar temperature conditions existed at the ends of heat runs three and five, as shown in Figure 24, and similar crack patterns at joint B and A, respectively, for those heated areas can be seen in Figure 23. Also, similar temperature conditions existed at the ends of heat runs four and six, as shown in Figure 24, and the absence of cracking can be noted for joints B and A, respectively.

It is apparent from the preceding discussion that problems with cracking can be anticipated, not only during periods of rapid cooling but also during periods of warmer temperatures that have been preceded by periods of low temperatures. One obvious approach, which may not be the most practical, is to heat treat only when ambient temperatures have been above some reasonable lower limit for a period of time. On the basis of the experimental data in Figures 24 and 25, it would appear that a reasonable lower temperature limit would be approximately 60°F (16°C) and that a reasonable time period for sustaining this temperature would be 1 day.

The cracks depicted in Figure 23 were visible only by very careful inspection of the deck surfaces. Also, as the slabs cooled to ambient temperatures the cracks apparently closed so tightly that they were no longer visible. This occurrence provides some indication of what performance may be expected of the cracked regions. Since the cracks are held so tightly by the reinforcing steel, it may be expected that they may heal themselves through continued hydration. If this proves to be the case, then their presence at this time would not be expected to hinder the performance of the decks.

Approximately 8 months after the heat treatment a white efflorescence was observed on the internally sealed decks along the crack locations. This was believed to be calcium carbonate formed from the carbonation of free lime in the concrete by carbon dioxide in the atmosphere. This process is probably beneficial to the condition of the cracks, because the calcium carbonate tends to fill void spaces where it occurs, and make the concrete less permeable to chemicals.(7) Similar though less frequent cracking with efflorescence was observed on the control spans, suggesting that at least part of the cracking on the experimental spans was due to plastic drying shrinkage and not to thermal effects.

A problem that has been noted on heat treated spans in some states has been the formation of minor spalls and cracks adjacent to the parapet. This problem resulted from difficulty in heating the parapets sufficiently so that lateral restraint could be minimized; during the heating operation minor spalls occurred along the face of the parapets and during the cooling period a shrinkage crack opened between the deck surface and the parapet face. This problem was avoided in the present study by not placing the parapets until after the heat treatments had been completed. Because the contractor elected to use precast parapets, there was only a single line of anchor bars protruding from the decks along the parapet locations. The electric blankets were easily placed longitudinally on either side of this steel to produce uniform heating along this boundary.

Cores 4 in. (102 mm) in diameter were drilled through cracks at two locations as indicated in Figure 25. In the polished sections of the cores shown in Figure 16 and 17, the cracks were traced with black ink to make them visible in the photographs. The nearly vertical cracks can be seen to be continuous through the overlay and base layer concrete.

The cracks in Figures 16 and 17 go around instead of through the coarse aggregate particles in the base layer concrete. This behavior is characteristic of plastic shrinkage cracks which occur due to rapid evaporation of water before the concrete has gained sufficient tensile strength to resist cracking. If the cracks in the base layer concrete did exist prior to placement of the overlay concrete, then the cracks through the overlay concrete may be considered as reflection cracks caused jointly by the preexisting weakness and by the heat treatment. The cracks through the overlay could, therefore, still be classified as thermal cracks.

Typically, plastic shrinkage cracks are not associated with accelerated deterioration of the deck areas where they occur, probably because these cracks are able to seal themselves through continued hydration of the cement particles adjacent to the crack interface. The thermal cracks, which are held tightly closed by the reinforcing steel, probably provide an even smaller opportunity for corrosive action than do the plastic shrinkage cracks.

Scheduled Deck Evaluations

Comparative evaluations of the three internally sealed concrete spans and the three adjacent control spans were scheduled to be made at the time of construction and after the first, third, and fifth winter seasons. The procedures to be performed are listed in Table 8 and include visual surveys and soundings on the deck surfaces, determination of the $C\ell^-$ concentrations in the concretes, and measurements of electrical potentials of the reinforcing steel. These procedures were performed five months after the deck was constructed and the results are discussed in the following sections.

Table 8

Scheduled Deck Evaluations at 0, 1, 3, and 5 Years

Decks		Procedure		
Type	No.	Visual Survey and Soundings	Chloride Sampling	Electrical Potentials
Internally Sealed	3	X	X	X
Conventional Construction	3	X	X	X

Visual Surveys and Soundings

The visual surveys conducted for the internally sealed and the conventional deck surfaces showed no defects. The thermal cracks, shown earlier in Figure 23, had closed tightly and were not visible to the unaided eye at the time of the survey. Hammer and chain drag soundings on the decks produced a ringing sound characteristically emitted by good bridge deck concrete.

Chloride Contents

The internally sealed concrete is intended to protect the upper reinforcing steel by preventing or seriously retarding the intrusion of chloride ions ($C\ell^-$) from deicing salt applications. Although the corrosion of steel embedded in concrete is dependent on several factors, including the presence of moisture and oxygen and the pH of the concrete, active corrosion has been reported to be suspected when the $C\ell^-$ concentration of a concrete reaches a threshold value of 1.3 $lb./yd.^3$ (0.77 kg/m^3).[8] In Virginia it has been found that varying amounts of $C\ell^-$ are contained in concretes due to naturally occurring $C\ell^-$ in the mix constituents, particularly the aggregates.[3] It is assumed that this natural, or background, $C\ell^-$ content does not contribute to the corrosion process and that it should, therefore, be measured initially and subtracted from future $C\ell^-$ determinations.

Samples of hardened concretes from each of the three experimental overlays and from each of the three conventional spans were analyzed for $C\ell^-$ contents. The powdered samples were tested for total $C\ell^-$ content using a potentiometric titration procedure with the Gran method of endpoint determination.[9] The average background $C\ell^-$ contents for these concretes are listed in Table 9.

Future evaluations of the bridge decks with respect to the rate of $C\ell^-$ intrusion will be made based on analyses of concrete samples drilled from the decks. This drilling will be performed using a rotary hammer with a 2-in. (51 mm) diameter bit, and the concrete samples obtained will be in a pulverized form. The occurrence of total $C\ell^-$ concentrations in excess of those listed in Table 9 will represent intruded $C\ell^-$ from deicing operations.

Table 9

Background Chloride Contents

Type Concrete	Chloride Content lb. $C\ell^-/yd.^3$ (kg $C\ell^-/m^3$)	
Internally Sealed	0.45	(0.27)
Conventional	0.68	(0.40)

Electrical Potentials

A copper-copper sulfate (CSE) half-cell was used to determine the electrical potentials of the reinforcing steel in the six study decks. The magnitude of such readings can be interpreted to indicate the probability of corrosion activity associated with the steel. Values from 0.00 to -0.20 volt CSE indicate a greater than 90% probability that no corrosion of the reinforcing steel is occurring; values in the range of -0.20 to -0.35 volt CSE indicate an uncertainty about the corrosion activity; and values of -0.35 volt CSE and beyond indicate a greater than 90% probability that corrosion of the reinforcing steel is taking place. The half-cell potentials of the reinforcing steel were measured in general conformance with ASTM C876 using a square grid pattern of 5 ft. (1.5 m). Permanent ground points were established for each experimental and control span by attaching electrical leads to the upper mat of reinforcing steel.

The electrical potentials were less negative than -0.20 volt CSE over 79% of the internally sealed spans, which suggests that no corrosion of the reinforcing steel is occurring. The electrical potentials from the conventional spans with epoxy coated reinforcing steel were slightly positive. This finding indicates the likelihood of a high electrical resistance in the circuit, which seems consistent with the insulating property of the epoxy coating.

CONCLUSIONS

An investigation of internally sealed concrete was conducted in the laboratory and on a two-course bridge deck and, on the basis of information presented in the report, the following conclusions were made.

1. The properties of the wax determined in the laboratory conformed to the properties recommended by the FHWA.

2. The cement factor had to be increased from 635 lb./yd.^{3} (377 kg/m^3) to 752 lb./yd.^{3} (446 kg/m^3) of concrete to achieve a 2 in. to 4 in. (51 mm to 102 mm) slump while maintaining a 0.47 water to cement ratio.

3. The dispersion of wax beads in hardened concrete was found to be uniform in both laboratory and field specimens.

4. Approximately 77% of the wax flowed into and sealed the paste regions surrounding the original locations of the wax beads.

5. The inclusion of a minimum of 4% to 6% entrained air in the fresh concrete is needed for adequate freeze-thaw resistance.

6. The average loss of compressive strength of the internally sealed concrete due to the heat treatment was approximately 5%.

7. The bond strength of the internally sealed overlay concrete to hardened base concrete was good without using a bonding agent and without roughening the base layer concrete.

8. The raked finish for the base layer can adversely affect the bond of the overlay concrete and is not recommended.

9. The effectiveness of the internal sealing was verified by water absorption tests and by the intrusion of chloride ions from a salt solution.

10. Cracking associated with the heat treatment of laboratory specimens was very minor in concrete containing the crushed limestone coarse aggregate approved for the project, but was significant in concrete containing a gravel coarse aggregate.

11. Some thermal cracking occurred as a result of the field heat treatments; however, it is believed that cracking of the bridge decks could be avoided by heating only entire spans and by heat treating only when ambient temperatures of 60°F (16°C) and higher have been sustained for 1 day.

12. The thermal cracks in the spans are very fine, being held together tightly by the reinforcing steel, and their presence is not expected to hinder the performance of the field installations.

13. The condition of the experimental and control spans is good as determined by visual surveys and soundings, electrical potential readings, and chloride ion contents; however some pattern cracks have appeared on both the experimental and control spans.

RECOMMENDATIONS

1. The concrete mixture had good placement characteristics and resulted in good properties for internally sealed concrete. It had a water to cement ratio of 0.47, a cement content of 752 lb./yd.^{3} (446 kg/m^3), a wax bead content of 114 lb./yd.^{3} (68 kg/m^3), and included 4% to 6% entrained air; a similar mixture should be used on any internally sealed decks.

2. Crushed limestone coarse aggregates may be suitable for use in internally sealed concrete, however gravel coarse aggregates are not recommended.

3. The heat treatment procedure should be modified to prevent thermal cracking by heating entire span lengths and by heating only when ambient temperatures of 60°F (16°C) and higher have been sustained for 1 day.

4. If the above recommendations can be implemented, internally sealed concrete should be considered an acceptable protective system for bridge decks.

5. The use of a screeded base layer without a raked finish is satisfactory for achieving good bond with the overlay concrete and should be specified for two-course construction.

6. The condition of the internally sealed and control spans should be monitored through evaluations at the ages of 1, 3, and 5 years. Test procedures should include visual surveys and soundings of the deck surfaces, determination of chloride contents of the concrete, and the measurement of electrical potentials of the reinforcing steel.

7. A decision concerning the use of internally sealed concrete should not be made until further evaluations can be made of pattern cracking that has appeared to varying degrees on both the experimental and control spans.

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8. Clear, K. C., and R. E. Hay, "Time-to-Corrison of Reinforcing Steel in Concrete Slabs", Federal Highway Administration, Report No. RD-73-32, April 1973.

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APPENDIX

VIRGINIA DEPARTMENT OF HIGHWAYS AND TRANSPORTATION
SPECIAL PROVISION FOR
INTERNALLY SEALED CONCRETE
Proj: 0064-003-104,B645 Rev. September 7, 1976

I. Description: This work shall consist of constructing bridge decks, as designated on the plans, in two separate stages including the furnishing, placing and finishing of the second stage deck surface concrete in accordance with this provision and in reasonably close conformity with the dimensions, lines and grades shown on the plans or established by the Engineer.

II. Plan of Operations - The Contractor shall furnish the Engineer a plan of operations relating to the work involved in the bridge deck placements for approval, prior to any placement operations. In preparing the plan, the Contractor shall consider that he shall not place the parapets until advised by the Engineer that the heat treatment process has been successfully completed.

This plan shall include batching operations, method of placement and proposed method of heat treatment. Approval of the Contractor's plan will not relieve the Contractor of his responsibility to meet all contract requirements.

III. Stage One construction shall consist of constructing the reinforced cement concrete bridge deck to the top of the upper mat of reinforcing steel, so as to provide at least 2 inches below the bridge deck finish grade for stage 2 construction, in accordance with Section 404 of the Specifications except as follows:

(a) The use of mechanical finishing screeds will not be required.
(b) The surface of the concrete shall have a roughened (raked) finish.
(c) Liquid membrane shall not be used for curing the concrete.
(d) Concrete shall not be treated with linseed oil.
(e) The requirement for nonpolishing fine aggregate will be waived.

IV. Stage Two construction shall consist of constructing the top 2 inches of the bridge deck with internally sealed concrete in accordance with Section 404 of the Specifications except as modified herein.

Stage two deck surface concrete shall be placed no sooner than 14 days or when stage one concrete has attained 85% of designed strength and not later than 60 days, unless otherwise approved by the Engineer.

The riding surface of the bridge deck shall have pronounced grooves approximately 1/8-inch deep by 1/8-inch wide. The groove pattern may be either 3/4-inch centers perpendicular to center line, or 3/4-inch centers parallel to center line with additional perpendicular grooves on 3 inch centers. Grooving shall be performed by mechanical means or other approved method.

(a) Materials - Section 219.02 of the Specifications is amended to include the following:

1. Wax Beads, used for internally sealing the concrete, shall be a blend of paraffin and montan wax conforming to the following requirements:

a. Physical Properties - Each bead shall be a physical blend of 75 ± 5 percent paraffin (melt point = 149° ± 2°F) and 25 ± 5 percent crude grade montan wax.

Bead size shall be as follows:

Passing 20 - mesh (0.033 inch) - 100 percent
Passing 80 - mesh (0.007 inch) - 0 to 5 percent

The beads shall be spherical in shape and shall, on the average, contain a void volume of 10 ±2 percent. No less than 60 percent of the beads shall have discernible voids.

The wax beads will be accepted under certification from the manufacturer that the beads meet the physical requirements stated herein.

b. Shipping and Storage - The wax beads shall be shipped and stored in a manner which will protect the beads from moisture and exposure to temperatures in excess of 120°F. The moisture content of the beads shall not exceed one (1) percent at any time prior to their introduction to the mix. A maximum temperature indicator shall be placed in one container of beads for each shipment lot location, and in each storage location at the site.

(b) Equipment shall conform to Section 404.19(f) of the specifications except as modified herein.

1. Finishing Machines and work bridge shall be capable of forward and reverse motion under positive control. Support rails will be required upon which all finishing machines and work bridge shall travel. Support rails shall be fully adjustable without the use of shims to obtain the correct bridge deck profile.

   The design of the finishing machines and appurtenant equipment shall be such that positive machine screeding and finishing of the plastic concrete shall be obtained within one inch of the curb line.

2. Vibrating and Oscillating Screed shall be used for the deck surface concrete. The screed shall be provided with positive control of the vertical position, the angle of tilt, and the shape of any deck crown. The screed shall be capable of or adequate provisions made for raising the screeding pan to clear the previously screeded surface when traveling in reverse.

   Construction joints shall be placed only where approved by the Engineer. The length of the screed shall be sufficient to extend at least 6 inches beyond an approved construction joint bulkhead, or the line where a construction joint saw out is approved, and shall overlap the edge of any previously placed section at least 6 inches. When placing concrete in a lane abutting a previously completed lane, that side of the finishing machine adjacent to the completed lane shall be equipped to travel on the completed lane.

3. Mechanical Strike-off screed or attachment will be required to provide a uniform thickness of deck surface concrete in front of the vibrating and oscillating screed.

(c) Proportioning of Materials - The mix shall be designed for an approximate 3% by weight bead content by reducing the amount of fine aggregate in the mix and shall conform to Section 219.07 of the Specifications, except that #7 or #78 aggregate shall be used for the overlayment mix in lieu of the #57's. as specified in Table II-11. The internally sealed concrete shall be non-air entrained.

(d) Mixing of Materials shall conform to Section 219.12 and 219.13 of the Specifications and the wax beads shall be introduced to the mix with the fine aggregate. Mixing shall then follow in the conventional manner.

The Contractor shall prepare a sufficient number of trial batches of the internally sealed concrete overlay prior to actual use, in order to determine any necessity for minor adjustments in the mix design, to allow the personnel to become familiar with the mixing and handling of the wax beads, and to ensure that the desired properties will be obtained.

(e) Placing and Finishing shall conform to Section 404.19(f) of the Specifications, except as modified herein.

Expansion joints shall be maintained through the overlayment.

Prior to placement of the deck surface concrete, all foreign material detrimental to achieving bond shall be removed by sandblasting. Formed edges at construction joints shall also be sandblasted to promote bonding. Sawn edges are not required to be sandblasted. At least one hour prior to placement of deck surface concrete, the stage one concrete surface shall be thoroughly water soaked. All puddles of standing water shall be removed prior to application of deck surface concrete.

Placement of the deck surface concrete shall be manipulated and mechanically struck off slightly above finished grade and then mechanically consolidated and screeded to finished grade with the vibrating and oscillating screed.

Upon completion of texturing operations, each section of the deck surface concrete shall be covered with wet burlap and polyethylene to prevent drying. Care shall be exercised to ensure that the wet burlap is well drained and that it is placed as soon as the surface will support it without deformation. Burlap and concrete surface shall be maintained in a continuous moist condition during the initial 24 hour curing period. After the initial 24-hour curing period, the remainder of the curing shall be in accordance with the Specifications. In the event liquid membrane is used for curing after wet curing, care shall be taken to assure complete coverage and that no shadowing occurs due to the surface texture.

The overlayment shall be placed the full length of the decks and full width between the innermost vertical reinforcing bars which extend up into the parapets.

(f) Heat Treatment - The overlayment shall be heat treated to melt the wax beads in the total depth of the overlay. The overlay shall be at least 21 days of age, shall be air dried for at least two days and shall have achieved its design compressive strength prior to heat treatment. The compressive strength is to be determined by tests made in accordance with AASHTO T22 and T23. Specimens shall be cured in a like manner as the overlay.

Heat treatment shall be accomplished in such a manner that the full depth of the overlay concrete attains a temperature of at least 185°F.

The concrete temperature at the surface shall not exceed 320°F. The heat treatment method shall be such that neither prolonged extreme temperatures nor open flames are subjected to the concrete which may cause surface deterioration.

The Department will furnish and install the necessary thermocouples and will monitor the temperature obtained by the Contractor's heat treatment.

The Department has arranged for the procurement of a blanket heat treating system from the FHWA along with at least one advisory person and such system will be made available at no cost to the Contractor. In the event the Contractor elects to utilize the FHWA system, he shall provide at his cost the following equipment:

1. A generator rated no less than 200 kw, 3 phase, 560 volts, 60 Hz and capable of sustained 200 kw loading. The generator shall be equipped with the following gages: cooling system temperature, voltage indicator, amperage indicator with rotary switch for checking current in each of 3 phases. Voltage control shall be easily accessible and variable from 420 v to 560 v.

2. Approximately 1800 sq. ft. of four to six inch unbacked fiberglass insulation batting. The insulation shall cover 100% of the treatment area and shall be maintained in good condition at all times.

3. Approximately 1272 sq. ft. of plastic mesh netting for retaining insulation in position.

4. Polyethylene sheeting for protection in the event of rain.

5. Lumber or rebars for weighting the insulation in place in the wind.

6. In the event it becomes necessary to walk on the insulation during heating, the Contractor shall provide suitable means to protect the insulation from damage.

7. Auxilliary 115 v, 15 amp power source for temperature indicators, electric drill and night lights.

V. Method of Measurement - Stage one will be measured and paid for in accordance with Sections 404 and 406 of the Specifications. Stage two internally sealed concrete overlayment will be measured in units of square yards.

VI. Basis of Payment - Stage two internally sealed concrete overlayment will be paid for at the contract unit price per square yard, which price shall be full compensation for surface preparation, heat treatment, furnishing and placing all material, and for all equipment, labor and incidentals necessary to complete the work.

Pay Item	Pay Unit
Internally Sealed Concrete	Square Yard

NOTES:

Technical advice and guidance relative to the wax beads may be obtained from Mr. R. L. Johnson of Interpace Corporation, ALPCO Wax Products, P. O. Box 785, Ione, California 95640, Telephone (209) 274-2444.

Technical advice relative to possible heat treatment methods may be obtained from Thermatronics Corp., 29 Dwight Place, Fairfield, New Jersey 07006.

Technical assistance will be available from VDH&T Research Council, FHWA and the manufacturer of the electric blankets.