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Number 14

Concrete Bridge Decks and

Pavement Surfaces

8 Reports

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HIGHWAY RESEARCH RECORD 14
CONCRETE BRIDGE DECKS AND PAVEMENT SURFACES

Contained in this 100-page book are 8 papers or reports presented at the 42nd Annual Meeting of the Highway Research Board, as follows:

"Effect of Insulating the Underside of a Bridge Deck," by E. O. Axon and R. W. Couch

"Experimental Roadway Heating Project on a Bridge Approach," by D. J. Henderson.

"Protection of Concrete from Deleterious Effects of Ice Removal Chemicals," by E. W. McGovern.

"History and Application of Glass Fiber Reinforced Resin Wearing Surface," by Raymond S. Blanchard.

"A Concrete Bridge Deck Survey by the SUR/FAX Photographic Method," by Harry E. Brown and Howard H. Newlon, Jr.

"Polyester Overlays for Portland Cement Concrete Surfaces," by L. E. Santucci.

"Environmental Influence on Corrosion of Reinforcing in Concrete Bridge Substructures," by J. L. Beaton and R. F. Stratfull.

"The Value of Insulated Forms for Winter Bridge Construction," by H. B. Britton.

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Contents

Effect of Insulating the Underside Of a Bridge Deck

E. O. AXON and R. W. COUCH, Respectively, Chief of Research and Research Engineer, Missouri State Highway Department

This study was designed to determine the merit of insulating the underside of a bridge deck in (a) preventing formation of ice or frost on a bridge deck before such formation on the abutting pavement, and (b) decreasing the number of freeze-thaw cycles and salt applications. Information is presented on application and bonding performance of the urethane foam, instrumentation, and collection and analysis of data for the periods of December 1, 1961 to April 29, 1962; and October 1, 1962 to November 30, 1962. Present data are considered insufficient to establish the merit of the insulation, but indicate that the effects tend to be beneficial.

• IN AUGUST 1961, the Bureau of Public Roads and the Missouri State Highway Commission approved an investigation designed to determine the effect of insulating the underside of a bridge deck. The outline of this investigation was as follows:

> In geographical areas subject to freezing temperatures it is well known to highway maintenance engineers that, under certain ambient weather conditions, there is a tendency for ice to form on bridge decks sometime prior to its formation on adjacent pavement. To traffic traveling on ice-free pavement this presents an oftentimes unexpected hazard. It also results in the number of applications of de-icing agents being considerably greater for bridge decks than for the adjoining pavement, which may be one of the factors in the earlier and more severe deterioration of the concrete in the decks as compared with that in the abutting pavement.
>
> It is desired to investigate the merit of insulating the underside of a bridge deck for:
>
> a. Preventing formation of ice on the bridge deck prior to such formation on the abutting pavement.
>
> b. Decreasing the number of freeze-thaw cycles and salt applications per year.
>
> To carry out the investigation it is proposed to insulate the underside of the deck of one of twin bridges, Number A-153, Route I-70, Cooper County, with a 3/4" thickness of urethane foam. Comparison of temperatures attained by, ice formation on, and deck deterioration occurring will be made between the insulated deck and the deck of the uninsulated twin bridge.
>
> Prior to application of the insulation, a pair of 10.00 ohm copper temperature coils will be installed in the deck, one being 1/4" below the upper surface and the other 1/4" above the bottom surface. Another pair of the coils will be similarly installed in the deck of the uninsulated twin bridge. Still another pair will be installed in the pave-

Paper sponsored by Committee on Effect of Ice Control.

ment approach slab, and a single coil will be rigged so as to sense the ambient air temperature.

The seven above listed temperature coils will be connected to an eight-point Leeds and Northrup Model S resistance-type temperature recorder (the eighth point will be attached to a zero-point check coil). The chart paper will have a range of -25° to 120°F. and a speed of four inches per hour. This instrument will measure and record the temperature at each point once every eight minutes, and provide a continuous printed record for as long as is desired.

From this record the following information may be obtained:

1. The temperature-differential between:
   a. The upper surface of the insulated deck, the uninsulated deck, and the pavement;
   b. Ditto for the bottom surface;
   c. The top and bottom surfaces of the three types of slab.
2. The time-lag between attainment of freezing temperature by each of the three types of slab;
3. The time-lag between attainment of thawing temperature by each of the three types of slab;
4. The number of freeze-thaw cycles undergone by each of the three types of slab.

In addition to the above information, it is proposed to:

1. Record the number of applications (and rate of each) of salt to the three types of slab;
2. During periods of decreasing temperatures, observe any differences in time of occurrence of icing on the three slabs; and also observe differences in time of melting;
3. Analyze three cores from each slab for mix composition, air content, and air-bubble size-distribution;
4. Make, and record in detail, semi-annual surveys of each slab for the time of appearance and the extent of any deterioration.

Finally, it is proposed to study the data and produce a progress report after each winter of exposure, and a final report when the latest records and observations seem no longer to be producing additional information. It is estimated that this point will be reached in three years.

The locations of all temperature coils are shown in Figure 1. The temperature coils were calibrated to 10.00 ohms at 77 F. All lead wires between the temperature coils and the recorder consisted of three-wire, solid copper conductor, thermostatic wire which provided temperature compensation of the leads. At the proposed locations for the temperature coils, the bridge decks were drilled with a 4-in. diamond bit, and the approach slab was drilled with a 6-in. steel shot drill. The coils were placed in fresh concrete ¼ in. from the surfaces where temperatures were to be recorded, except in the insulated deck where the coil over the I-beam was placed at the same depth from the top surface as the bottom coil between the I-beams.

The coil over the I-beam in the bottom of the insulated deck was installed to determine if the half-depth insulated I-beam would affect the temperature of the concrete above it. As this coil was not included in the original outline, it could not be connected to the eight-point recorder unless it was substituted for another. Initially, it was hoped that the coil could be substituted for the zero-point check coil. However, inability to frequently check the operation of the recorder made it mandatory to keep the zero-check coil in operation, as the zero-check coil permitted corrections of recorded temperatures when the chart paper shifted during long periods of unchecked operation of the recorder.

Application of the urethane foam was made during November 1961 by use of a spray gun and a platform hoist. Two separately heated chemicals were fed to the mixing

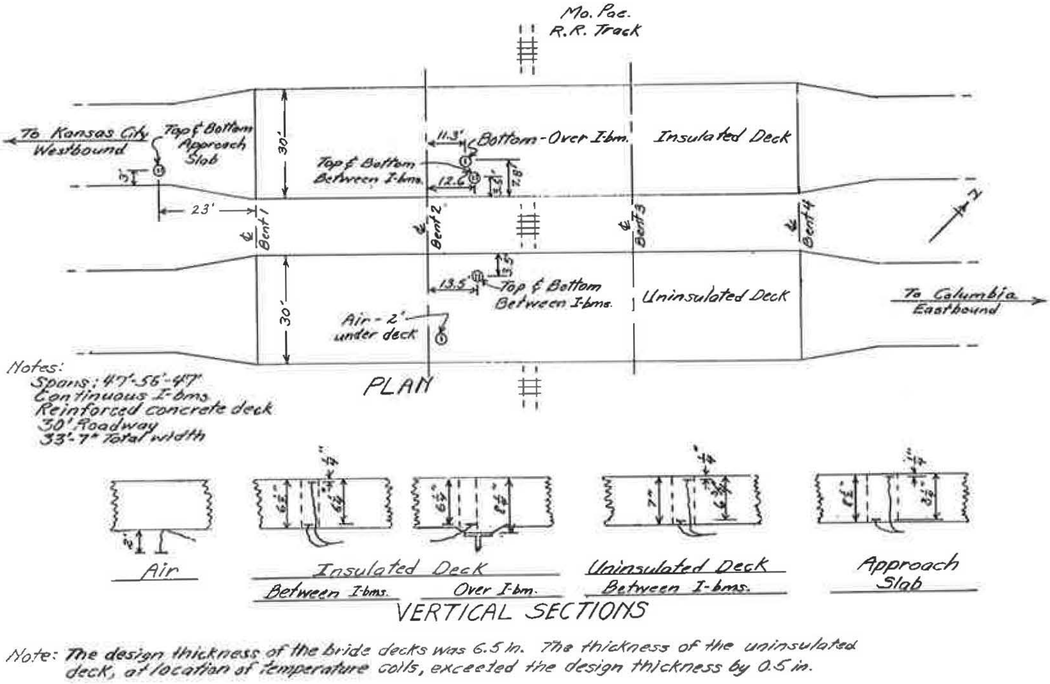

Figure 1. Layout of copper temperature coils in decks of bridge A-153.

chamber of the spray gun where they were combined and sprayed on the surfaces. Heating equipment consisted of electric belts and immersion coils for barrels, and electric line coils for the hoses from the chemical pump to the spray gun. The two chemicals reacted best to form a good foam when the ambient air temperature was above 60 F and the relative humidity of the air was low. On several occasions work was stopped because of cold weather or high humidity. Each of these appeared to prevent the proper reaction from taking place. An attempt was made to apply a thin coat of chemicals to cold surfaces for the purpose of insulating subsequent sprays from cold surfaces. This was discontinued when it was found that thickness of the final foam varied greatly, frequent clogging of the spray gun occurred, and appreciable quantities of the chemicals were wasted. It took seven days to apply the urethane foam on approximately 790 sq yd of surface, including the time lost due to inclement weather and temporary malfunctioning of equipment. It is estimated that the work could have been done in four or five days in good weather.

Initially, the urethane foam appeared to bond satisfactorily to both concrete and steel, inasmuch as during application only a few small areas had to be stripped and resprayed because of poor bond. However, during the early part of the winter some evidence of poor bond was observed between the foam and the webs of the aluminum painted I-beams. The results of a semi-detailed survey made in January 1962 indicated the following:

1. The bond between the foam and the concrete was still satisfactory.
2. Some evidence of poor bond was observed along approximately 65 percent of the total length of the stringers.
3. There was no correlation between air temperature at time of application and loss of bond.
4. The loss of bond was probably caused by condensations, on the stringers, of exhaust fumes from diesel locomotives.

The contractor removed and replaced the foam in areas of poor bond in September 1962.

Originally it was anticipated that recording of temperatures would start on October 1, 1961, before freezing temperatures occurred. However, the delay in application of urethane foam prevented the start of the temperature record until December 1, 1961. Consequently, the temperatures reported and discussed are for the periods of December 1, 1961 to April 29, 1962; and October 1, 1962 to November 30, 1962, inclusive.

In studying these results, several methods were used. This was necessary because none of the methods gives a complete picture regarding the over-all effect of the insulation. This is at least partly due to the fact that the insulation not only delays the time of freezing but also delays the time of thawing. This complicates the study because in evaluating the results there are at least three icing conditions that could affect the interpretation of the results:

1. The formation of frost on bridge decks and not on pavements on days when no precipitation occurs.
2. The time of formation of ice or nonmelting of snow during periods of precipitation.
3. The time of melting of ice or snow during thawing periods.

Obviously, any decision regarding insulation of bridge decks will depend on the effect of the insulation during each condition. That the effect of the insulation could be beneficial under some but not all of these conditions is also obvious. Further complicating any decision regarding the benefits of the insulation is the effect of the use of de-icers.

Although present data are definitely insufficient to warrant any conclusions regarding the merits of insulating a bridge deck, the methods used in studying the results were designed (a) to provide the maximum number of indications from available data, and (b) ultimately to provide the necessary information, from these and future data, to permit an evaluation of the merits of insulating a bridge deck.

The following methods were used in studying these results:

1. Determination of the number of freeze-thaw cycles at the various locations (Table 1).

TABLE 1

NUMBER OF FREEZE-THAW CYCLES[a,b]

Month	Freeze-Thaw Cycles (no.)							
	Air	Uninsul. Deck		Insul. Deck			Approach Slab	
		Top	Bottom	Between I-Beams Top	Between I-Beams Bottom	Over I-Beam, Bottom	Top	Bottom
Dec. '61	9	11	9	9	7		15	3
Jan. '62	8	13	9	11	9		15	6
Feb. '62	13	19	14	18	15		20	9
Mar. '62	11	16	14	13	12	11	11	
Apr. '62	7	7	7	6	6	3	3	
Oct. '62	2	3	2	2	2	1	0	
Nov. '62	8	12	7	10	7	4	3	
Total	58	81	62	69	58	19	67	18

[a]32 F considered positive and not freezing.
[b]From December 1, 1961 to April 29, 1962; and October 1, 1962 to November 30, 1962.

2. Determination of the relationship between the minimum daily temperatures at the various locations, with results shown in Table 2 for the tops of the decks and approach slab.

3. Determination of the relationship between the maximum daily temperatures at the various locations, with results shown in Table 3 for the tops of the decks and approach slab.

4. Determination of the percent of time the temperature of the air and the tops of the slabs was below 32 F (Table 4).

5. Relationship between occurrence of freezing and thawing in the air and the tops of decks and approach slab (Table 5).

6. Determination of the relationship between time of start of freezing and thawing (Table 6).

TABLE 2

RELATIONSHIP BETWEEN MINIMUM DAILY TEMPERATURES OF TOPS OF SLABS VS AIR

Location of Temp. Coil	Month		Number of Days When Min. Temp. of Point with Respect to Min. Air Temp. Was					
			Less Than		Same As		Greater Than	
			Days	%	Days	%	Days	%
Top approach slab	Dec.	1961	6	19.4	4	12.9	21	67.7
	Jan.	1962	8	25.8	2	6.5	21	67.7
	Feb.	1962	5	17.9	2	7.1	21	75.0
	Mar.	1962[a]	4	13.3	3	10.0	23	76.7
	Apr.	1962[b]	0	0.0	0	0.0	29	100.0
	Oct.	1962	0	0.0	1	3.2	30	96.8
	Nov.	1962	1	3.3	2	6.7	27	90.0
Total			24	11.4	14	6.7	172	81.9
Top insulated deck	Dec.	1961	14	45.2	5	16.1	12	38.7
	Jan.	1962	19	61.3	3	9.7	9	29.0
	Feb.	1962	8	28.6	4	14.3	16	57.1
	Mar.	1962[a]	11	36.7	6	20.0	13	43.3
	Apr.	1962[b]	4	13.8	5	17.3	20	68.9
	Oct.	1962	8	25.8	6	19.4	17	54.8
	Nov.	1962	18	60.0	5	16.7	7	23.3
Total			82	39.0	34	16.2	94	44.8
Top uninsulated deck	Dec.	1961	14	45.2	11	35.5	6	19.3
	Jan.	1962	22	71.0	4	12.9	5	16.1
	Feb.	1962	14	50.0	9	32.1	5	17.9
	Mar.	1962[a]	20	66.7	7	23.3	3	10.0
	Apr.	1962[b]	12	41.4	7	24.1	10	34.5
	Oct.	1962	21	67.7	4	12.9	6	19.4
	Nov.	1962	25	83.3	1	3.3	4	13.4
Total			128	61.0	43	20.4	39	18.6

[a]Temperatures not recorded on March 21.
[b]Temperatures not recorded on April 30.

TABLE 3

RELATIONSHIP BETWEEN MAXIMUM DAILY TEMPERATURES OF TOPS OF SLABS VS AIR

Location of Point (Temp. Coil)	Month		Number of Days When Max. Temp. of Point with Respect to Max. Air Temp. Was					
			Less Than		Same As		Greater Than	
			Days	%	Days	%	Days	%
Top approach slab	Dec.	1961	4	12.9	3	9.7	24	77.4
	Jan.	1962	5	16.1	1	3.2	25	80.7
	Feb.	1962	3	10.7	3	10.7	22	78.6
	Mar.	1962[a]	1	3.3	3	10.0	26	86.7
	Apr.	1962[b]	1	3.4	1	3.4	27	93.2
	Oct.	1962	0	0.0	0	0.0	31	100.0
	Nov.	1962	0	0.0	3	10.0	27	90.0
Total			14	6.7	14	6.7	182	86.6
Top insulated deck	Dec.	1961	9	29.0	6	19.3	16	51.7
	Jan.	1962	7	22.6	5	16.1	19	61.3
	Feb.	1962	1	3.6	2	7.1	25	89.3
	Mar.	1962[a]	0	0.0	2	6.7	28	93.3
	Apr.	1962[b]	0	0.0	0	0.0	29	100.0
	Oct.	1962	0	0.0	0	0.0	31	100.0
	Nov.	1962	2	6.7	1	3.3	27	90.0
Total			19	9.0	16	7.6	175	83.4
Top uninsulated deck	Dec.	1961	4	12.9	4	12.9	23	74.2
	Jan.	1962	6	19.3	1	3.2	24	77.5
	Feb.	1962	2	7.1	2	7.1	24	85.8
	Mar.	1962[a]	0	0.0	3	10.0	27	90.0
	Apr.	1962[b]	0	0.0	1	3.4	28	96.6
	Oct.	1962	0	0.0	1	3.2	30	96.8
	Nov.	1962	2	6.7	0	0.0	28	93.3
Total			14	6.7	12	5.7	184	87.6

[a]Temperatures not recorded on March 21.
[b]Temperatures not recorded on April 30.

7. Determination of the probable maximum number of occurrences of frost on the concrete surfaces (Table 7).
8. Relationship between the number of salt applications on the insulated and uninsulated decks (Table 8).

These results could be discussed at great length. However, a thorough discussion of these preliminary data is considered to be unwarranted, because as previously stated these data are considered insufficient to warrant any conclusions regarding the merits of insulating a bridge deck. Therefore, in this progress report these data are being presented, together with the following brief listing of the most obvious indications:

TABLE 4

PERCENTAGE OF TIME THAT AIR AND CONCRETE WERE BELOW 32 F

Month	Percentage of Time Below 32 F			
	Air	Top Uninsulated Deck	Top Insulated Deck	Top Approach Slab
Dec. 1961	78.9	77.8	81.0	76.6
Jan. 1962	80.8	76.7	78.7	84.2
Feb. 1962	52.1	48.5	42.7	40.7
Mar. 1962	33.2	27.4	23.4	22.2
Apr. 1962	7.0	7.5	5.1	2.0
Oct. 1962	1.1	2.5	1.7	0.0
Nov. 1962	8.6	9.9	9.9	2.3
Avg.	37.6	36.0	35.0	32.9

Table 1

1. The number of freeze-thaw cycles in the tops of the decks and approach slab exceeded the number of cycles in air (81, 69, and 67 vs 58).

2. Of the three points in the tops of the decks and approach slab, the greatest (81) and the least (67) number of cycles were obtained in the uninsulated deck and the approach slab, respectively. However, the number of cycles in the top of the insulated deck was only two greater than the number in the top of the approach slab.

3. Comparison of the freeze-thaw cycles in the bottom of the insulated deck over a half-depth insulated I-beam and midway between I-beams (for March, April, October, and November) shows that the cycles in the bottom of the deck over the I-beam were fewer than those midway between I-beams (19 vs 27). This indicates that more than half-depth insulation of an I-beam is necessary to eliminate the effect of the I-beam on the temperature of the concrete above it. However, it is presently indicated that the concrete over the I-beam tends to be warmer. As this latter indication might be different during the colder months, the temperature of this point will be continuously recorded during the winter of 1962-63. This, of course, means that the temperature of the bottom of the approach slab will not be recorded during the winter of 1962-63.

Table 2

1. The minimum temperature of the top of the uninsulated deck was lower than the minimum air temperature on 61 percent of the 210 days. This indicates that the minimum temperature in the top of the uninsulated deck tended to approach that of a wet bulb.

2. The minimum temperature of the top of the approach slab was higher than the minimum air temperature on 81.9 percent of the 210 days. This shows the beneficial effect of heat from the subgrade.

3. The average relationship between the minimum temperatures of the top of the insulated deck vs air tends to be approximately midway between that of the top of uninsulated deck vs air and that of the top of approach slab vs air. However, the results obtained for the insulated deck varied appreciably between months. In fact, the results obtained for the insulated deck approached that obtained for the uninsulated deck during December, January, and November. Therefore, these data indicate that the beneficial effect of the insulation was variable.

TABLE 5

RELATIONSHIP BETWEEN OCCURRENCE OF FREEZING AND THAWING IN AIR AND TOPS OF DECKS, AND APPROACH SLAB

Point 1	Point 2	Month	No. of Cycles Freeze and Thaw Occurring				
			Concur-rently	In (1) When (2) Was		In (2) When (1) Was	
				Frozen	Thawed	Frozen	Thawed
Air	Top insul. deck	Dec.	7	2	-	2	-
		Jan.	7	-	1	3	1
		Feb.	13	-	-	5	-
		Mar.	8	-	3	5	-
		Apr.	6	-	1	-	-
		Oct.	2	-	-	-	-
		Nov.	8	-	-	-	2
Total			51	2	5	15	3
Air	Top uninsul. deck	Dec.	9	-	-	2	-
		Jan.	7	-	1	5	1
		Feb.	13	-	-	6	-
		Mar.	11	-	-	5	-
		Apr.	7	-	-	-	-
		Oct.	2	-	-	-	1
		Nov.	8	-	-	-	4
Total			57	-	1	18	6
Air	Top approach slab	Dec.	9	-	-	6	-
		Jan.	7	1	-	6	2
		Feb.	12	-	1	8	-
		Mar.	6	-	5	5	-
		Apr.	3	-	4	-	-
		Oct.	-	-	2	-	-
		Nov.	3	-	5	-	-
Total			40	1	17	25	2
Top insul. deck	Top uninsul. deck	Dec.	8	1	-	3	-
		Jan.	11	-	-	2	-
		Feb.	18	-	-	1	-
		Mar.	13	-	-	-	3
		Apr.	6	-	-	-	1
		Oct.	2	-	-	-	1
		Nov.	10	-	-	-	2
Total			68	1	-	6	7
Top insul. deck	Top approach slab	Dec.	8	1	-	7	-
		Jan.	10	1	-	3	2
		Feb.	17	-	1	3	-
		Mar.	11	-	2	-	-
		Apr.	3	-	3	-	-
		Oct.	-	-	2	-	-
		Nov.	3	-	7	-	-
Total			52	2	15	13	2
Top uninsul. deck	Top approach slab	Dec.	11	-	-	4	0
		Jan.	11	2	-	2	2
		Feb.	18	-	1	2	-
		Mar.	11	-	5	-	-
		Apr.	3	-	4	-	-
		Oct.	-	-	3	-	-
		Nov.	3	-	9	-	-
Total			57	2	22	8	2

TABLE 6

COMPARISON OF TIME PRECEDENCE OF FREEZING AND THAWING

Point 1	Point 2	Month	Cycles of Time Precedence During: Freezing, 1 Precedes	Freezing, Same	Freezing, 2 Precedes	Thawing, 1 Precedes	Thawing, Same	Thawing, 2 Precedes
Air	Top uninsul. deck	Dec. 1961	4	1	4	2	0	6
		Jan. 1962	4	1	2	2	1	5
		Feb. 1962	8	1	4	4	1	7
		Mar. 1962	5	0	6	5	0	7
		Apr. 1962	3	0	4	3	1	3
		Oct. 1962	0	0	2	2	0	0
		Nov. 1962	5	2	1	4	1	3
Total			29	5	23	22	4	31
Air	Top insul. deck	Dec. 1961	4	0	3	5	0	1
		Jan. 1962	6	0	1	5	0	3
		Feb. 1962	11	0	2	4	1	7
		Mar. 1962	5	1	2	2	0	7
		Apr. 1962	5	1	0	3	0	3
		Oct. 1962	1	0	1	2	0	0
		Nov. 1962	7	0	1	8	0	0
Total			39	2	10	29	1	21
Air	Top approach slab	Dec. 1961	3	2	4	2	0	6
		Jan. 1962	3	0	4	3	1	4
		Feb. 1962	7	1	4	4	1	6
		Mar. 1962	4	0	2	0	0	7
		Apr. 1962	3	0	0	0	0	3
		Oct. 1962	0	0	0	0	0	0
		Nov. 1962	3	0	0	0	0	3
Total			23	3	14	9	2	29
Top uninsul. deck	Top insul. deck	Dec. 1961	1	1	6	7	0	0
		Jan. 1962	6	1	4	11	1	0
		Feb. 1962	16	1	1	7	2	8
		Mar. 1962	12	1	0	11	3	3
		Apr. 1962	6	0	0	3	0	0
		Oct. 1962	2	0	0	2	0	0
		Nov. 1962	6	1	3	9	0	1
Total			49	5	14	50	6	12
Top uninsul. deck	Top approach slab	Dec. 1961	6	1	4	1	2	7
		Jan. 1962	0	2	9	4	3	5
		Feb. 1962	14	1	3	3	2	12
		Mar. 1962	8	0	3	1	1	10
		Apr. 1962	3	0	0	0	0	3
		Oct. 1962	0	0	0	0	0	0
		Nov. 1962	3	0	0	0	0	3
Total			34	4	19	9	8	40
Top insul. deck	Top approach slab	Dec. 1961	5	0	3	1	0	6
		Jan. 1962	0	1	9	0	0	11
		Feb. 1962	6	2	9	2	1	13
		Mar. 1962	2	1	8	0	0	12
		Apr. 1962	3	0	0	0	0	3
		Oct. 1962	0	0	0	0	0	0
		Nov. 1962	3	0	0	0	0	3
Total			19	4	29	3	1	48

TABLE 7

PROBABLE MAXIMUM NUMBER OF OCCURRENCES OF FROST ON CONCRETE SURFACES

Data from Table	Probability of Frost		Possible Number of Occurrences of Frost on		
	High[1]	Low[2]	Uninsulated Deck	Insulated Deck	Approach Slab
All Data for Seven Months					
5	Yes	--	6	3	2
6	Yes	--	23	10	14
High prob. total			29	13	16
5	--	Yes	0	2	1
6	--	Yes	22	29	9
Low prob. total			22	31	10
Grand total			51	44	26
Excluding Data for December, January, and February[3]					
5	Yes	--	5	2	0
6	Yes	--	13	4	2
High prob. total			18	6	2
5	--	Yes	0	0	0
6	--	Yes	14	15	0
Low prob. total			14	15	0
Grand total			32	21	2

[1]Condition occurs (a) during periods of falling temperatures, and (b) at night or near dawn.
[2]Condition occurs (a) during periods of rising temperatures, and (b) usually between 7:00 AM and 5:00 PM (daytime).
[3]Because of frequent applications of de-icing salts.

TABLE 8

NUMBER OF SALT APPLICATIONS 1961 - 1962

Date	Number of Salt Applications							
	Uninsulated Deck				Insulated Deck			
	Light	Moderate	Heavy	Condition	Light	Moderate	Heavy	Condition
Dec. 8	0	1	3	Snow	0	1	3	Snow
9	0	2	1	Snow	0	2	1	Snow
11	0	0	8	Snow	0	0	8	Snow
12	0	0	3	Snow	0	0	3	Snow
18	0	0	4	Ice	0	0	4	Ice
19	0	5	0	Ice	0	5	0	Ice
22	0	3	0	Snow	0	3	0	Snow
23	0	2	1	Snow	0	2	1	Snow
Jan. 5	0	3	1	Ice	0	3	1	Ice
6	0	5	2	Snow	0	5	2	Snow
7	0	3	1	Snow	0	3	1	Snow
14	0	1	2	Snow	0	1	2	Snow
15	0	2	1	Snow	0	2	1	Snow
21	0	2	4	Ice	0	2	4	Ice
22	2	0	5	Ice	2	0	5	Ice
23	0	0	2	Ice	0	0	2	Ice
Feb. 20	0	2	1	Snow	0	2	1	Snow
23	0	1	5	Snow	2	1	3	Snow
24	0	3	0	Snow	0	3	0	Snow
Total applic.	2	35	44		4	35	42	

Table 3

1. During the colder months of December and January, the maximum temperature of the top of the insulated deck tended to be slightly lower than that of either the top of the approach slab or the top of the uninsulated deck. This indicates that during these two months the insulation tended to delay the warming up of the concrete, and should delay the melting of accumulations of ice and snow.

2. During February, March, April, and October, this trend was reversed for the bridge decks. This indicates that during four of the seven months' accumulations of ice and snow should melt slightly faster on the insulated than on the uninsulated deck.

Table 4

1. During the colder months of December and January, the temperature of the top of the insulated deck remained below 32 F for a slightly higher percentage of time than did that of the top of the uninsulated deck.

2. During February, March, April, and October, the percentage of time that the top of the insulated deck remained frozen was less and greater, respectively, than that for the tops of the uninsulated deck and the approach slab.

3. In general, the air temperature remained below 32 F a greater percentage of time than did the tops of the decks and approach slab.

Table 5

1. The number of concurrent freeze-thaw cycles of any two points (locations) was less than the number of freeze-thaw cycles at either comparative point (location). This shows that freeze-thaw cycles did not always occur concurrently at two locations.

2. The number of concurrent freeze-thaw cycles in air and the top of each of the three slabs was 51, 57, and 40 for the insulated deck, uninsulated deck, and approach slab, respectively. The difference in number of concurrent cycles was primarily due to the amount of heat available, in or beneath the concrete in each slab, to prevent freezing of the concrete (and moisture thereon) when the air temperature was below 32 F. These data, therefore, indicate that the available heat was greatest in the approach slab and least in the uninsulated deck.

3. In the next to last column, 15, 18, and 25 cycles occurred in the top of the insulated deck, the top of the uninsulated deck, and the top of the approach slab, respectively, when the air remained below freezing. The difference in number of cycles was primarily due to the availability of heat to the concrete, either by absorption from the sun or from below, to cause thawing of the concrete (and ice or snow thereon) when the air temperature was below 32 F. These data indicate that the available heat was greatest in the approach slab and least in the insulated deck.

4. Frost on the concrete surfaces would only be considered probable when the temperature of the concrete was lower than that of the air. For the noncurrent cycles this condition existed when a cycle was obtained in air while the concrete remained frozen, and when a cycle was obtained in the concrete while the air temperature remained above freezing (thawed). The first condition occurred 2, 0, and 1 times, and the second 3, 6, and 2 times, respectively, on the tops of the insulated deck, uninsulated deck, and approach slab.

5. In comparing the cycles in the tops of the insulated and uninsulated decks, it is apparent that the difference in number of cycles obtained at these two points was caused by the uninsulated deck cooling down and warming up faster than the insulated deck. Six and seven cycles were obtained in the top of the uninsulated deck when the top of the insulated deck remained frozen and thawed, respectively.

Table 6

These comparisons are for the concurrent cycles shown in Table 5.

1. In the majority of concurrent freeze and thaw cycles the air temperature preceded the concrete temperatures during freezing and lagged the concrete temperatures

during thawing. Under this condition, frost on the concrete surfaces would be considered impossible.

2. Frost on the concrete surfaces would be considered possible when the concrete temperature preceded the air temperature during freezing, or lagged the air temperature during thawing. The first condition occurred 23, 10, and 14 times, and the second 22, 29, and 9 times, respectively, on the tops of the uninsulated deck, insulated deck, and approach slab.

3. In comparing the time precedence of freezing and thawing of the tops of the uninsulated and insulated decks, it is evident that in general the top of the uninsulated deck preceded the top of the insulated deck in both freezing and thawing. This indicates that the insulation should tend to delay the formation and melting of ice on the bridge deck.

Although not shown by these data, the effect of the insulation on the time of freezing tended to exceed the effect on time of thawing.

Table 7

These comparisons combine the indicated possible occurrences of frost for the nonconcurrent cycles in Table 5 and the concurrent cycles in Table 6.

The possible maximum number of occurrences of frost for the seven months were 51, 44, and 26 on the uninsulated deck, insulated deck, and approach slab, respectively. However, the probability of frost forming on the concrete surfaces is dependent on factors other than the concrete temperature being lower than the air temperature. For example (Table 7) a low probability of occurrence of frost is expected during periods of thawing or rising temperatures which normally occur in the daytime. If these low probability occurrences be deducted from the above maximum seven months' totals, only 29, 13, and 16 high probability occurrences remain for the uninsulated deck, insulated deck, and approach slab, respectively. This indicates that the highly probable occurrences of frost on the insulated deck were fewer than those on either the approach slab or the uninsulated deck.

In addition, during months of frequent salt applications, a low probability of occurrence of frost would be expected for all conditions. Therefore, by excluding the data for December, January, and February only 18, 6, and 2 high probability occurrences of frost remain for the uninsulated deck, insulated deck, and approach slab, respectively. This indicates that during the months of March, April, October, and November, the highly probable occurrences of frost on the insulated deck were one-third those for the uninsulated deck, and three times those for the approach slab.

Table 8

The total number of salt applications (81) was the same for the insulated and uninsulated decks. However, on February 23, the amount of salt applied to the insulated deck was less than that applied to the uninsulated deck. All salt applications were applied to remove snow and ice, and no applications were made because of frost on the decks. Furthermore, all salt applications were made during the three colder months. Precipitation during March, April, October, and November was light and no salt applications were required. If frost occurred on these concrete surfaces during March, April, October, and November, it apparently was insufficient to require applications of salt.

These data indicate that salt applications were related to (a) the time of occurrence and amount of precipitation, and (b) the temperatures of the concrete surfaces. Therefore, during certain periods, the effect of time of occurrence and amount of precipitation could overshadow the effect that insulation may have on the amount of salt needed.

At the time when insulation was applied to the underside of one of these decks, both surfaces were in good condition with the exception of a few short hairline cracks on each deck adjacent to curbs. During the winter of 1961-62, one partial transverse crack and a moderate amount of surface mortar deterioration occurred on each deck. Tests on three full depth cores from each bridge deck indicated that the surface mortar

TABLE 9

RESULTS FROM TESTS OF THREE FULL DEPTH CORES FROM EACH BRIDGE DECK

Type Deck	Core No.	Percent Air	Cement Factor	W/C Ratio (by vol.)
Insulated	61-340	4.3	1.53	0.66
	61-344	1.9	1.62	0.89
	61-345	4.6	1.62	0.71
Uninsulated	61-341	3.8	1.60	0.72
	61-342	4.8	1.47	0.74
	61-343	2.4	1.71	0.71

deterioration occurred in areas where insufficient air was entrained in the concrete. The results on these six cores are given in Table 9.

SUMMARY

This first progress report of effect of insulating the underside of a bridge deck covers the periods of December 1, 1961 to April 29, 1962; and October 1, 1962 to November 30, 1962. These periods are definitely of insufficient length to warrant conclusions regarding the justification of the insulation. However, the following indications were obtained:

1. The minimum temperature of the top of an uninsulated bridge deck tends to be lower than that of either the top of the approach slab or air.
2. Insulating the underside of a bridge deck tends to produce the following:
    a. Reduction of the number of freeze-thaw cycles.
    b. Reduction of the severity of freezing by raising the minimum temperature.
    c. Delay of the start of both freezing and thawing.
    d. Reduction of the length of time the concrete is frozen, because the delay in start of freezing tends to be greater than the delay in start of thawing.
3. With respect to formation of frost or ice on the deck the insulation should tend toward the following:
    a. Decrease in the occasions when frost forms on the bridge deck but not on the approach slab.
    b. Delay in the formation of ice on the deck. In general, but with exceptions, it would be anticipated that ice would form first on the uninsulated deck, next on the insulated deck, and last on the approach slab. This order should prevail during moderate and intermittent cold periods, but could be reversed during severe and prolonged cold periods.
    c. Delay during extremely cold periods and hastening during moderately cold periods the melting of accumulated ice and snow.
4. With respect to applications of salt needed to keep the deck free of frost or ice, the insulation could be both detrimental and beneficial. During severe and prolonged cold periods, more salt could be required; during moderate and intermittent cold periods, less salt could be required. Based on the record of salt applications during the periods of observation, the insulation did not significantly affect the amount of salt applied.
5. With respect to amount of concrete deterioration occurring during the periods of observation, the insulation had no significant effect.

Experimental Roadway Heating Project On a Bridge Approach

D. J. HENDERSON, Supervising Electrical Engineer, Bureau of Electrical Operations, New Jersey State Highway Department

During October 1961, electric heating cables for snow removal and ice control were installed in connection with a bituminous concrete resurfacing operation on the approach to a State Highway drawbridge in Newark, New Jersey. The cables were installed in an 840-ft length of two lanes of the bridge approach roadway. This approach ascends at a 3 percent grade and was the scene of major traffic delays during the heavy snowstorms of the previous winter. The installation was made under emergency conditions on a very heavily traveled highway and the work of laying the cable was designed so as to produce minimum delay or interference with resurfacing operations.

This paper describes methods and special equipment used for laying the heating cable and the various steps of the entire operation. Installation costs and complete design data are given including watts per square foot dissipation, total power required, conductor sizes, and electric circuitry. The winter of 1961-62 in the area was very mild with little snow accumulation. This condition precluded the gathering of firm data on performance. Conclusions are given concerning suitability of materials and methods of construction, together with a brief analysis of electric power costs as related to the rate structure of the utility company. Photographs of several steps in the construction are included.

• THE DISRUPTIONS of highway traffic caused by the heavy snows in New Jersey during the winter of 1960-61 focused attention on the necessity to study the possibility of improving methods of snow removal. The first result of these studies was the identification of critical locations where traffic had stalled on grades early in the storms, making it impossible to get plows into the area to clear the roadway.

One of these trouble spots was the westerly approach to the large movable bridge spanning the Passaic River on Routes US 1 and 9 in Newark. This approach was the scene of a major traffic tie-up during the storm of December 12, 1960. The 1,000-ft length of this approach rises at a grade of 3 percent. The average daily traffic volume is over 50,000, with heavy trucks constituting 40 percent of the total. The substantial grade, the presence of heavy and rapid traffic, and the storm tie-up combined to recommend this approach as a proper location for an experimental roadway heating project. Fortunately, a large supply of power was available, already installed, providing for the operation of the drawbridge.

Search for precedents for such an installation revealed that an apparently successful installation of electric heating cables has been made on the Mound in Edinburgh, Scotland. The heating units were installed in conjunction with an asphalt resurfacing project. In Aberdeen, S. D., electric heating cables covering only the 18-in.-wide wheel tracks, were installed in the concrete roadway of an overpass. Both these installations are apparently doing a satisfactory job, although neither is associated with a heavily traveled public highway. An electric heating installation in such a highway would be a pioneering effort on a purely experimental basis.

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It was early recognized that the major problem was one of installing the heating cable in a resurfacing operation without damaging the conductors and without serious delay or disturbance to the paving work. To obtain some information on the effect of a normal resurfacing operation on an insulated electrical conductor, test lengths of mineral-insulated cable were subjected to conditions that would prevail during such an operation, passage of heavy trucks and caterpillar treads of paving machines over them, and the compaction of bituminous material around them. After a series of these operations, the test lengths were removed, examined for sleeve damage, and tested for insulation resistance. Results indicated that this type of cable would withstand practically any amount of pressure from truck tires, caterpillar treads, or the compaction of bituminous material, provided a sharp stone did not press directly upon the cable sleeve.

With this background information, the installation was planned in four steps:

1. Laying a coarse-aggregate leveling course.
2. Laying the heating cables and securing them to the leveling course.
3. Spreading a ½-in. coat of sand-mix asphalt by hand to cover the cables; rolling with a 10-ton roller.
4. Laying the 1½-in. final course by paving machine in the usual manner.

The portion of this roadway selected for heating traverses both a land fill area and a bridge area. All available data, including that from the Edinburgh and South Dakota installations indicated that a heat dissipation of 30 watts per sq ft would be sufficient for the land fill area, and 40 watts per sq ft for the bridge area. There are no firm data to indicate how much heat would be lost by conduction downward, though there appears to be a consensus that the heating cables should not be installed more than 2 in. below the roadway surface. Using the accepted value of the heat of fusion of ice, and assuming that the installation will be required to perform satisfactorily when snow is accumulating at the rate of 1 in. per hr, a dissipation of 40 watts per sq ft gives nearly twice as much heat as is required to melt 1 in. of snow per hour at 32 F. If the extreme assumption is made that one-half the heat is conducted downward, then the remaining half is sufficient for the purpose on the bridge area when the benefit of the ef-

TABLE 1

DESIGN DATA FOR INSTALLATION OF HEATING CABLE[a]

Design Factor	Land Fill Area	Bridge Area
Length (ft)	710	130
Width (2 lanes) (ft)	20	20
Area (sq ft)	14,200	2,600
Watts per sq ft dissipated	30	40
Total power required (kw)	426	140
Nominal voltage, 3φ, 3 w (v)	450	450
Watts per ft of heater cable	11.2	15
Heater cable, total length (approx.) (ft)	38,000	8,400
Heater cable spacing (in.)	4-5/16	3-3/4
Approx. calc. length of unit (ft)	1,410	945
Actual length of unit to fit length of area (ft)	1,420	910
Conductor size, heater cable	No. 14 AWG	No. 16 AWG
Conductor size, cold lead	No. 6 AWG	No. 6 AWG
Type of cable	Mineral insulated	Mineral insulated
Depth in cover (in.)	± 2	± 2

[a]Portion of roadway heated, 2 right-hand lanes; total length heated, 840 ft; and total area heated, 16,800 ft.

fects of heavy traffic are taken into consideration. Conditions on the land fill area are not so critical and it was assumed that a somewhat lesser rate of heat dissipation would be sufficient.

With $1\frac{1}{2}$- to 2-in. cover over the heating cables, it was felt that there might be a possibility of creating a plane of cleavage if the cables were spaced too closely. A minimum spacing of $3\frac{1}{2}$ in. was fixed. Holding to this minimum spacing required that the conductors should dissipate 11 to 15 watts per linear foot, a rather high figure for plastic-insulated cable. This high heat dissipation, combined with the requirement for mechanical strength of cable sheath, determined the selection of mineral-insulated cable.

Three-phase, three-wire power at 450 volts was available from the bank of 600-KVA transformers feeding the bridge. This amount of available power fixed the minimum area of roadway that could be heated. For economy in power distribution, it was desired to keep the number of heating units as low as possible and this consideration ruled out the use of resistance alloy heating elements. All these controlling factors pointed to the use of a single copper conductor carrying current sufficient to raise the resistance loss to the desired wattage. Calculations on this basis indicated the use of No. 14 B& S gage copper on the land fill area and No. 16 on the bridge area. All units were star-connected in groups of three to the three-phase, 450-volt line. A grounded neutral was desirable, but the establishment of such a neutral was not practical.

Figure 1 shows the physical characteristics of the roadway area involved. Figure 2 shows the placement of the heating cables. The photographs, taken during the work of cable installation and paving, give a clear idea of the methods used.

For convenience, limitation of voltage drop and economy of distribution copper, the cold ends of the units on the land fill area were all brought out in a slot cut in the roadway directly over the bridge abutment. Each 1,420-ft unit was laid in a "U," 710 ft long, bringing both ends of the "U" into this slot. One cold end of each of the 910-ft bridge area units was laid in this slot and the other end taken down through a hole in the deck 130 ft away, at the end of the heating area on the bridge. This procedure was followed because it was necessary to make seven 130-ft passes for each unit and this odd number of passes made it impossible to bring both ends of the bridge units back to the slot.

On the land fill area, the conductors were laid in two groups: 14 units in the first pass and 13 in the second. As the conductors payed out from the reels on the cable-laying rig (Fig. 3), they were cemented to the roadway with asphalt joint sealer every 4 ft. On the bridge area, the single conductor was payed out from a hand-operated reel around templates at each end of the bridge area. These templates had semicurcular

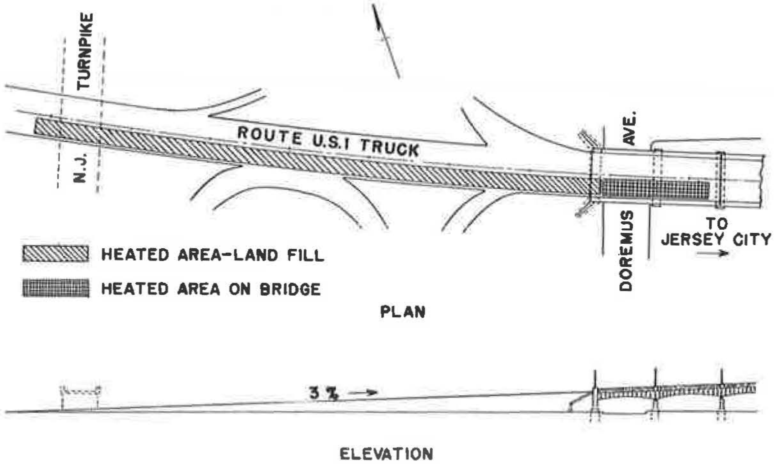

Figure 1. Test site.

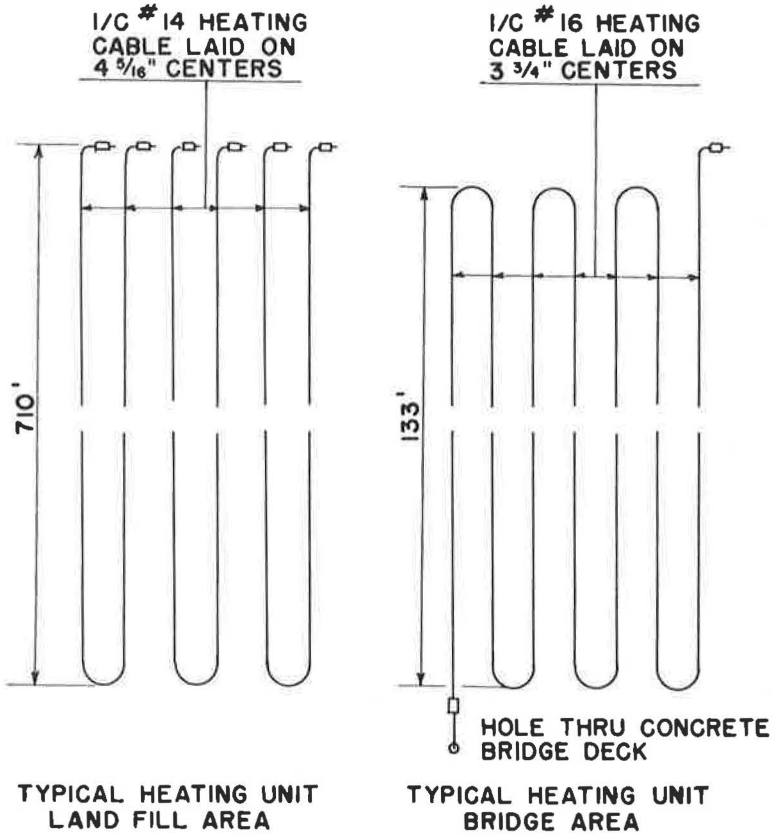

Figure 2. Layout of heating cables.

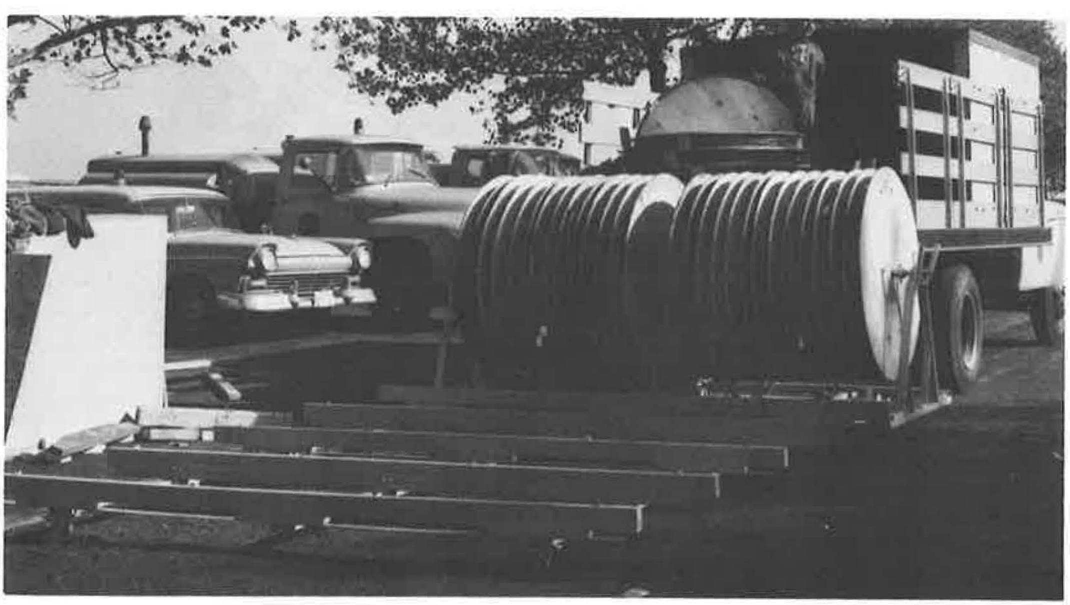

Figure 3. Cable-laying rig.

lugs of the size and spacing required to maintain the 3¾-in. spacing of the cable (Fig. 4). Each unit was secured to the pavement with asphalt joint sealer before the templates were removed.

After the laying of the leveling course, the work of cable laying and paving was done on two consecutive weekends; the land fill area was completed on the first weekend and the bridge area on the second. In each case, the planned steps were followed and the entire operation went smoothly. There was no indication that the paving operations did any damage to the heating cable.

The mechanics of laying the cable on the land fill area with the cable-laying rig were relatively simple (Fig. 5). Each of the 27 reels carried one 1,420-ft unit wound double; that is, in an elongated "U." The unit had been wound on the reel starting with the closed end of the "U" and terminating with the cold conductors.

The function of the harrow-like assembly or drag on the cable-laying rig was to straighten the cables, take the curvature or kinks out of them, and align them in the proper spacing. This drag consisted of a series of split yokes mounted on a rigid frame. Each yoke consisted of a piece of 2- by 4-in. lumber with clearance holes for the cables at the proper spacing. The piece was split down the center line of the holes and the halves held together with wing bolts. These yokes were opened and the cables drawn out from the reels and laid in the grooves of the bottom halves of the yokes. The top halves were then closed and secured with the wing bolts.

To begin the actual laying process, a similar split yoke was anchored to the roadway at the edge of the roadway slot. When the cable-laying rig was in position for the start of the run, the yoke was opened and the ends of the conductors placed in the proper grooves in the bottom half of the yoke, with the splicing sleeves and cold conductors on the slot side of the yoke. The yoke was then closed, securing the cable ends in position by reason of the bearing of the splicing sleeves against the fixed yoke.

The rig was started and moved down the road at the rate of 5 to 10 ft a minute. The work of securing the cables to the binder course followed about 30 ft behind the laying rig at the point where the cables lay naturally on the roadway surface. At 4-ft intervals, a piece of 2- by 6-in. timber was laid across the cables, holding them firmly against the surface of the road while the asphalt joint sealer was poured (Figs. 6 and 7).

Figure 4. Laying cable, land fill area.

Costs of this installation were accumulated in such a manner as to segregate the cost of paving from the cost of laying the cable and installing electrical equipment. The cost of laying the cable and the installation of electrical distribution and control equipment was $25,777. This electrical equipment does not include any transformers or service facilities. These facilities were already installed to serve the drawbridge, and power for the roadway heating installation was taken from the bus bars of the bridge switchboard through suitable circuit breakers and interlocking mechanisms. The cost does include the construction of the cable-laying rig and the installation of temporary lighting required because most of the work was done at night. The figure represents a cost of $1.56 per sq ft for the 16,800-ft area. With the knowledge that this cost is burdened by factors that accompany any experimental installation, and further, with expenses incurred by the doing of the work under emergency conditions on a heavily traveled roadway, it is estimated that under normal conditions the same installation could be made at a maximum cost of $1.10 per sq ft.

As just stated, the cost of this installation does not include the cost of transformer and electric service equipment. The cost of such equipment for a load of this size would be in the neighborhood of $18,000. With this included, the unit cost of the entire installation, under normal conditions, would be $2.19 per sq ft, and it is apparent that one-half of this cost would be absorbed in transformer and power supply equipment. In other words, the cost of transformer and switching facilities ahead of the switchboard would be approximately equal to the cost of the installation of the heating cable and the electrical distribution facilities on the load side of the switchboard. There is every indication

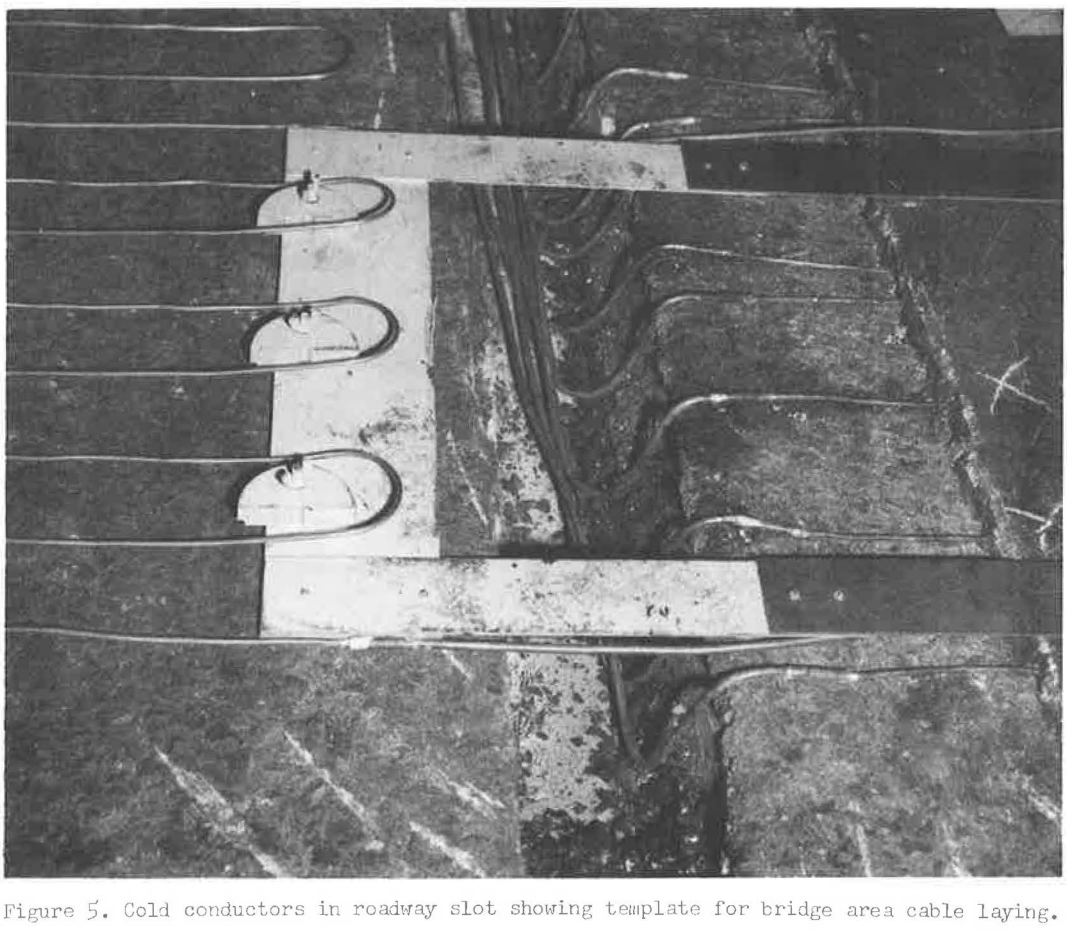

Figure 5. Cold conductors in roadway slot showing template for bridge area cable laying.

Figure 6. Spreading sand-mix asphalt.

Figure 7. Rolling sand-mix.

that this ratio will persist, generally, as the size of the project increases, up to the point where power requirements are too great to be served from the power company's 4,100-volt primary network.

Figures on the energy costs for heating this 16,800-sq ft area of roadway can be misleading, unless the utility rate structure is understood. The 600-KVA service for the bridge carries a monthly minimum charge of $400. On the condition that the roadway heating supply would be interlocked at the bridge switchboard in such a manner that the bridge operation and the heating operation could not be carried on simultaneously, the utility company agreed that there would be no charge in this monthly minimum. The remainder of the monthly bill is composed of the demand charge and the consumption charge. By the very nature of its use, a roadway heating load has a high demand, with relatively low consumption during any month of operation. It has been estimated that in the New York Metropolitan area a roadway heating system would be used only 125 hr in any one winter. If a heating system were used for 2 or 3 hr for only one storm in any one month, the demand charge for that month would be established, although the consumption charge would be relatively small. This unbalanced relationship between demand and consumption imposes the same economic penalty on roadway heating as on any large, intermittent load.

The roadway heating system was used during the period from November 8, 1961 to February 7, 1962, for a total of 67.7 hr. This use covered three monthly billing periods for energy consumed. Each billing, of course, was for a total consumption of energy by bridge operation and roadway heating. For the purpose of identifying the approximate cost of energy for the roadway heating, it is sufficiently accurate to compare the billing for these periods with the billing for similar periods in the preceding year. The difference in these billings would represent the increase in cost due to the use of roadway heating.

Although Table 2 does not furnish a full picture of the cost of operation of this heating system during a severe winter, nor the cost of operation of this system if it were installed and fed independently of any other load, it does show very strikingly the overriding influence of the demand charge in the energy cost picture. For this reason, figures on costs per hour have little actual significance, except to show that, for any one billing period, the hourly cost will vary inversely with the period of use. Rate structures of utility companies may show wide variations in different parts of the country. With knowledge of the total load and the rate structure of the utility involved, the energy cost for any heating installation can be readily calculated.

The entire bridge approach was resurfaced, but heating cables were installed in only one-third of the area. Altogether, 1,127 tons of bituminous concrete were used, ex-

TABLE 2

OPERATIONAL COSTS

Billing Period	Total Bill ($)	Cost of Heating ($)	Hours Used	Cost per Hour ($)
11-8/12-8-61 11-8/12-8-60	1,231.98 490.90	741.08	4	$185.75
12-8-61/1-10-62 12-8-60-1- 9-61	1,627.51 481.62	1,145.89	28.75	39.85
1-10-62/2-7-62 1- 9-61/2-9-61	1,561.44 489.62	1,071.75	34.95	30.66
Total		2,958.72	67.7	
Avg.				43.70
Avg. per sq ft				0.0026

clusive of the material in the leveling course. Of this total, only 6 percent was spread by hand raking. There is no doubt that the hand-placing operation increased the unit cost of paving, but because of the very small quantity involved, it is impossible to determine this increase with any degree of accuracy. During the times when the hand-spreading operation was being performed in the cable area, paving crews were also engaged in paving operations elsewhere on the project, absorbing what would otherwise have been idle time. The paving crew was augmented to accomplish the hand-laying operation. A survey of this additional manpower indicates that hand raking added approximately $0.50 per ton to the placement of the material. This cost increment applies only to the 68 tons of sand-mix. When the total cost of the entire resurfacing operation is considered, the added cost of placing the sand-mix is insignificant.

In the ten months since this installation was completed, there has been no evidence that the heating cables have tended to lift or separate the resurfacing from the leveling course. It is, of course, too early to assume that this will not happen, though the relatively wide spacing of the cables makes it highly improbable. It was anticipated that the cables might tend to rise out of the sand-mix ahead of the roller during compaction, but this operation did not appear to alter the position of the cables in any manner. Contrary to expectations, rising of the heating cable did occur at two places while the final course was being placed by the paving machine. When the machine stopped for a few minutes during change of trucks for reloading, the cable rose out of the loose material directly behind the paving machine. This was attributed to the fact that the stopping of the paving machine permitted a concentration of heat in the area underneath the machine hopper which produced excessive expansion of the cable at that point. This buckling of the cable did not occur at any time when the paver was in continuous operation, nor did it occur during the final compaction process.

Unfortunately for the success of this experiment, snowfall in the Newark area during the winter of 1961-62 was very light. There were no storms that were characterized either by low temperatures, heavy precipitation, or long duration. Only one storm began with air temperature as low as 26 F and road surface temperature 25 F. Atmospheric temperature during this storm rose rapidly. This rise in temperature, coupled with the action of traffic, made it impossible to evaluate the effect of the heating system. No storm was of sufficient duration to permit the roadway temperature to reach a steady state. It is therefore impossible to make a firm evaluation of the ultimate capabilities of this system from the necessarily fragmentary data accumulated. Visual observations tend to confirm expectations that the heat dissipations, 40 and 35 watts per sq ft for the bridge and land fill areas, respectively, are sufficient to melt 1 in. of snow per hour, but this could not even be tentatively documented by instrumentation during the brief, light snowfalls of the 1961-62 winter.

Although documentation on performance of this installation must await the winter of 1962-63, some conclusions concerning materials, methods, and other details of this experimental installation can be reached.

1. An electric heating cable installation can be made in conjunction with a bituminous concrete resurfacing operation without undue delay or interference with the resurfacing work.

2. The hand placing and rolling of the intermediate sand-mix course does not make a significant increase in the cost of resurfacing if the work operations are properly staged.

3. No change in design of the cable-laying rig is indicated. A slight alteration in the bridle attachment to the truck is indicated to accommodate small changes in direction where cables pass around manholes or catch basins.

4. The method of securing the cables to the tack coat immediately after laying can be improved to eliminate the possibility of disturbance of the alignment of the cables by workmen during hand spreading of the sand-mix. A covering of cheesecloth saturated with asphalt joint sealer is being considered.

These conclusions necessarily relate only to the mechanical aspects of this installation. Optimum electrical design could not be attained because of the limitations imposed by the type and characteristics of the existing electrical service. Mineral-insulated cable appears to be well suited for this use, but there is no reason to assume that this is the only type of insulated cable that should be considered. The main virtue of this experiment lies in its proof of the ease with which heating cable can be included in a resurfacing operation on a major highway.

This leaves only the problem of economic justification. Heavy power costs, by reason of high demand and infrequent use, appear to limit these installations to relatively small critical areas where traffic delays and hazards caused by snow or ice justify such costs. Ease of installation and low maintenance cost of a system of electric cable heating recommend its use, but a wide area of investigation remains to determine the most effective and economical use of electric power. One area for study is the feasibility of continual heating of the roadway at a low dissipation rate throughout the winter season and stepping up this rate when required by storm conditions. This procedure might operate to level demand charges and make the load more desirable to utility companies, particularly if it is combined with some other load such as highway lighting.

Economic losses resulting from traffic delays and accidents in snowstorms on heavily traveled highways demand that research in roadway heating be uregently pursued. This experimental project helps to remove one of the problems from the area of speculation.

Discussion

J. D. GEORGE and IAN DYKE, Metropolitan Toronto Roads Department—The author's report was interesting and easy to follow. This was probably due to the fact that in the snow-melting experiment for the F.G. Gardiner Expressway in Toronto, Canada, the Metropolitan Toronto Roads Department is meeting similar problems.

The reasons for the conception of the project, which the author has clearly indicated in the first part of his report (i.e., important traffic routes, long, steep grades, and a location difficult to keep free of snow by conventional means) are all basic requirements for such a project. If these conditions were not present it would not be possible to justify using electricity to melt snow and ice. If any one of these three conditions were lacking, it would be difficult to justify such an expensive means of roadway heating.

The design of the author's project differs in many respects from the F.G. Gardiner Expressway project and a few others with which the writers are familiar. The Passaic River project uses mineral insulated cable for heating element, long panel stretches, and high voltages. Ordinary steel wire mesh (without insulation), short panel stretches, and low voltages (maximum 30 v) for safety have been used in experiments.

In one case, expensive heating element is combined with low-cost switch gear and transformers; in the other case, a low-cost heating element is combined with high-cost switch gear and transformers. No doubt each system has its merits and should be used to suit local conditions and reduce over-all capital costs.

The author's report does not indicate the means by which the power supply is controlled in his project, nor does he give any indication of the time required from when the power is turned on to when the pavement surface is warm enough to melt snow. Because the scheme does not use a base load to keep the pavement warm throughout the winter, this time element is quite important and could become critical during operation.

Construction aspects of the author's project seem to follow the usual trend as far as we know. Heating element is generally placed in the asphaltic surfacing of the roadway approximately 2 in. below the driving surface. Spacing of the heating elements seems to be generally between 2 and 4 in. However, the Metropolitan Toronto Roads Department is considering using, in a future experiment, a special layer of 2-in. fine-aggregate portland cement concrete in which to embed the heating element and then top it with waterproofing and the usual $1\frac{1}{2}$-in. asphaltic concrete wearing course.

As for the economics of the method, the writers fully agree with the author's conclusions that this type of heating can be justified only under the extreme conditions mentioned.

Protection of Concrete from Deleterious Effects of Ice Removal Chemicals

E. W. McGOVERN, Technical Representative, Tar Products Division, Koppers Company, Pittsburgh, Pa.

• A MAJOR maintenance problem facing highway departments in the snow and ice belts of the northern hemisphere is keeping roads and streets in condition for the safe movement of traffic throughout the year. The significance of this maintenance problem becomes more apparent, in this era of rising prices, when highway departments have to stretch maintenance budgets to satisfy the demands of the motoring public.

Prior to World War II, plowing, sanding, or cindering was considered adequate for ice and snow control. Since then, the public has pressured highway departments for roads clear of ice and snow. The most widespread means employed is the use of chemicals, principally sodium chloride and calcium chloride. However, the use of chlorides has created a serious problem of scaling or spalling of portland cement concrete because of their deleterious effects.

Numerous methods have been used to protect portland cement concrete against the actions of these chemicals. Often the line of investigation was determined by evaluation of products with which the individual investigator was already familiar. This investigation is no exception. This report describes some experiences using a coal tar compound to protect portland cement concrete from the harmful effects of chlorides.

The problem of maintaining concrete highways and structures against the harmful effects of ice control and other chemicals is not new. Forty years ago Lord (1) investigated the practicability of protecting concrete with tar against alkali waters. His investigations showed that portland cement concrete of good quality could be protected against alkali attack by coating with water gas tar and coal tar. In all of his investigations the protection to portland cement concrete was increased when the concrete was coated with water gas tar followed by a coat of coal tar.

LABORATORY INVESTIGATION

After World War II, when the problem of salt action on cement concrete was becoming evident, an investigation was started to determine the effectiveness of various tar compounds to protect portland cement concrete. The fact that coal tar in conjunction with water gas tar was more effective than water gas tar alone suggested the investigation of various coal tar compounds combining the penetrating properties of water gas tar with the protective properties of coal tar.

In 1957 a blend having the typical test properties given in Table 1 was developed and test-marketed for protecting concrete. The product is marketed under the name "Concrete Sealer." The protective characteristics of this coal tar blend were evaluated in the laboratory using the following procedures on prepared specimens of air-entrained and non-air-entrained PC concrete.

Effects of Calcium Chloride Solutions

Tests on Air-Entrained Concrete.—In the investigation on the effects of $CaCl_2$ solution, PC concrete slabs (24 × 12 × 2 in.) made with air-entrained portland cement were purchased from a local construction company and stored for six months before using.

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TABLE 1

TYPICAL TEST PROPERTIES OF CONCRETE SEALER

Determination	Value
Specific gravity at 25/25 C	1.122
Water (% by vol.)	1.8
Specific viscosity, Engler, 50 cc at 40 C	2.4
CS_2 insoluble (% by wt.)	3.13
ASTM D-20 distillation	
(% by wt.) to 170 C	0.9
200 C	2.6
235 C	16.9
270 C	38.6
300 C	50.1
Softening point of residue above 300 C (R & B °C)	39.0
Specific gravity at 15.5/15.5 C, distillate to 300 C	1.027
Sulfonation index, total distillate (% by wt.) to 300 C	0.1

From information furnished by the supplier, the slabs consisted of one part type 1-A portland cement, two parts sand, and three parts pea gravel and had a minimum strength of 4,000 psi.

Surface Penetration by 20% $CaCl_2$ Solution.—For the surface penetration test a slab was prepared in the following manner:

The surface on one-half of the slab was coated with 0.1 gal per sq yd of sealer and allowed to cure for 24 hr, whereas the surface on the remaining half was left uncoated.

On each half a cylinder made from a quart can coated with coal tar pitch was sealed to the surface and filled to a depth of 4½ in. with a 20% $CaCl_2$ solution then allowed to stand at room temperature (Fig. 1).

Every two days solution was added to the can on the untreated section to bring the level to 4½ in. The level of the solution in the cylinder on the coated section remained constant and the treated portion of the slab remained dry. After two weeks, the concrete on the surface and edges of the untreated section started to disintegrate. After 15 months, when the test was discontinued, the treated section was still intact, whereas the untreated section showed measurable disintegration.

Immersion in 20% $CaCl_2$ Solution.—For the immersion test a concrete slab was prepared in the following manner:

The slab was cut in half. One half was completely coated with 0.1 gal per sq yd of sealer and allowed to air cure in the laboratory for 24 hr. At the end of 24 hr both halves were immersed in shallow (3 in. deep) pans in a 20% $CaCl_2$ solution (Fig. 2). The solution covered the slabs to a depth of ½ in. The pans were

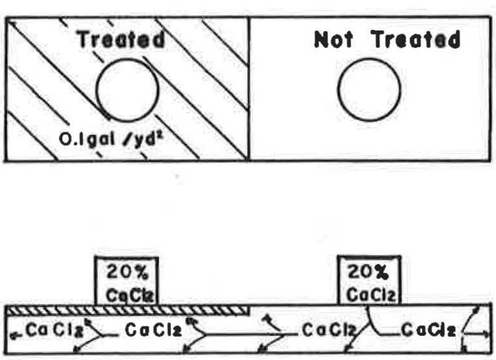

Figure 1. Surface penetration 20% $CaCl_2$ solution into treated and non-treated air-entrained concrete.

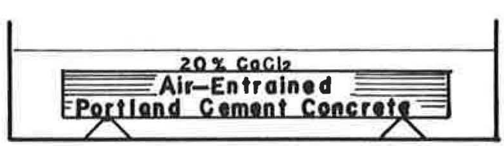

Figure 2. Immersion in 20% $CaCl_2$.

left uncovered in the laboratory at room temperature and the solution was allowed to evaporate and crystallize. Every 14 days tap water was added to bring the solutions back to the original depth. At the end of 14 days the uncoated section had started to disintegrate; after 140 days it had almost completely disintegrated. After 280 days, when the test was discontinued, the coated section was still intact with no signs of distress.

Tests on Non-Air-Entrained Concrete. —In the investigation on the effects of dilute solutions of NaCl and $CaCl_2$, under freezing-and-thawing conditions, concrete mortar blocks were prepared following the procedure described in ASTM Test Method D 1191-52-T. After curing, the blocks were stored for six months before using.

Freezing and Thawing in 3% Solutions of NaCl and $CaCl_2$. —Some of the blocks were treated by immersing in sealer for a few minutes at room temperature, then allowed to air cure 24 hr at room temperature. The treated and untreated blocks were placed in porcelain pans and immersed in 3% solutions of NaCl and $CaCl_2$. The pans were placed in a cold box at -20 F for 16 hr, removed, and the solutions allowed to thaw for 2 hr. The samples were then removed from the solutions and allowed to air-dry at room temperature for 6 hr. This procedure was repeated for 15 cycles.

At the end of 5, 10 and 15 cycles of freezing and thawing, the blocks were examined for signs of spalling. After 5 cycles, the untreated blocks showed signs of spalling. After 15 cycles the surface of the untreated blocks was completely pitted, whereas the treated blocks remained intact with no signs of distress.

FIELD APPLICATIONS

The sealer has been used experimentally by several State highway departments and toll road authorities. It has been marketed commercially in New York, Connecticut, Pennsylvania, Ohio, and the District of Columbia.

Primarily it has been used to protect new portland cement concrete. However, it has also been used to treat new or spalled portland cement concrete prior to surfacing with a hot plant mix or surface treatment.

New York

In 1957 the Town of Greece, N. Y., reported a serious maintenance problem. New PC concrete depressed curbing deteriorated during the first winter under the action of rock salt used for ice control. Different kinds of sealers and protective coatings had been tried, with generally unsatisfactory results. The newly developed sealer was offered, and used to treat 8 miles of new PC concrete depressed curbing and 1 mile of PC sidewalks. After six years the concrete is still sound, with no signs of spalling or scaling due to the action of salt. The untreated portland cement concrete curbing failed after one winter due to the deleterious effects of salt.

Connecticut

In December 1957 Connecticut treated several PC concrete bridge decks that were poured after October 1, 1957, on the Connecticut Turnpike, east of the Branford Toll Station. These decks have been subjected to ice removal chemicals through six winters, with no signs of scaling or spalling on the treated sections.

Pennsylvania

In late November 1959, a week before being opened to traffic, an area of approximately 12,000 sq yd on the Oil City Bypass on Pa. 62 was treated with 0.10 gal per sq yd. The application was made during 40 F weather. Right after application, rain started and continued for several days. This was the first time that the sealer was subjected to rain before curing occurred. Apparently the rain did not interfere with the protective properties of the sealer. The treated pavement has gone through three winters, with no evidence of scaling. Inspection in September 1962 showed no evidence of scaling, whereas an adjacent untreated pavement exhibited several small areas (5 to 10 sq yd each) that were scaling.

On December 11, 1959, new PC concrete totaling approximately 35,000 sq yd—on the interchange of US 20 with the Erie Thruway and on Pa. 89 at Northeast, Pa.—was treated with 0.10 gal per sq yd of sealer. Shortly after application of the sealer started, a freezing rain mixed with snow and sleet began, then later changed to snow. The project engineer decided to complete the treatment, because the forecast was for snow and he wanted the new pavement to have some protection before using salt. These treatments have been subjected to three winters with no evidence of spalling or scaling.

In October 1960, two weeks prior to opening Interstate 90 to traffic, about 55,000 sq yd of new PC concrete pavement were treated with 0.07 to 0.08 gal per sq yd of sealer. After two winters there is no evidence of spalling or scaling on the treated areas. However, a number of bridge decks on Interstate 90 that were not treated have spalled considerably from the effects of ice control chemicals. Several of these bridge decks have been repaired at a cost of approximately $3 per square yard.

In November 1960, just prior to opening to traffic, approximately 18,000 sq yd of new PC concrete on a section of US 119, known as Homer City Bypass, were treated with 0.072 gal per sq yd of sealer. This section of highway borders on a spray pond used for water aeration and much of the time the pavement is damp. Whenever the temperature drops below freezing, the pavement is salted twice a day. This treated pavement has been through two winters with no signs of scaling. On the areas not treated, the concrete is spalling and scaling.

In December 1961, 7,000 sq yd on a new concrete bridge deck at Tionesta, Pa., were treated with 0.053 gal per sq yd of sealer. This bridge was subjected to frequent treatments of salt and sand during the 1961-62 winter, with no harmful effects to the concrete.

OTHER FIELD APPLICATIONS

The sealer has also been used to treat spalled concrete before resurfacing with hot plant mix or surface treatment and to treat new PC concrete pavement prior to surfacing with hot plant mix.

In 1958 an opportunity arose to use the sealer on several short sections on the New York Thruway near Utica that had disintegrated to the point that they were unsafe and required extensive rehabilitation to restore the riding surface and prevent further damage to the concrete. These damaged sections were treated with 0.20 gal per sq yd of sealer, then surfaced with a treatment of RT-10 and N. Y. No. 1 (½ in.–¼ in.) chips or N. Y. 1-A (¼ in.–⅛ in.) chips. After three years, the treatment was still intact, with no indication of further disintegration of the damaged concrete.

In 1959 the Ohio Turnpike reported a serious maintenance problem at Exit 11. As cars decelerated then stopped at the toll gate extra amounts of salt solution dripped off the cars, causing spalling. To prevent further deterioration of the pavement, and to prevent further action of ice removal chemicals, the area was treated with 0.1 gal per sq yd of sealer, then covered with a hot plant mix (to restore the smooth riding surface). After three winters there are no signs of further deterioration.

During 1961 and 1962 several new PC bridge decks and overpasses in Washington, D. C., were treated with two applications of 0.1 gal per sq yd of sealer, followed by an application of coal tar pitch emulsion, prior to surfacing with hot plant mix. It is still too early to determine the long-time effectiveness of these treatments.

GENERAL COMMENTS

When to Apply Sealer

Field and laboratory tests were conducted to determine the best time to apply the sealer to portland cement concrete. Observations showed that the sealer applied to new clean portland cement concrete penetrated and dried within 24 hr after application. In the field, this indicated that the best time to treat is before the pavement is opened to traffic. Highway traffic tends to fill the pores in PC concrete pavement with dirt, rubber, oil and grease. However, the sealer can be applied to concrete pavement after opening to traffic. In one instance, 12 miles of PC concrete pavement 3 to 5 years old were treated with the sealer material, which usually penetrated and dried within 24 hr.

Method of Application

The sealer can be applied through standard bituminous distributors, by tack coat (spray) equipment, by sprinkling cans, or by brush. It is a thin material with a viscosity of 5 to 30 centipoises at 100 F. When applied through a bituminous distributor, the distributor tank, lines, and spray bars should be cleaned before adding the sealer, which will cut and loosen any bituminous residue, thus giving an unsightly appearance to the pavement. In addition, the spray bar should be covered with a hood to prevent the fog spray from being carried by the wind. This is especially important in built-up areas. The usual rate of application varies from 0.04 to 0.10 gal per sq yd. General practice has been to apply (at 125 to 150 F) about 0.05 gal per sq yd of sealer when no pretests have been performed to determine optimum application rates. Preferably, pretests should be made on the pavement or structure that is to be treated.

Color Characteristics

The initial color of portland cement concrete pavement treated with the sealer varies from light brown to deep black. This is a temporary condition; after several months, traffic wears off the surface color and within 12 to 18 months the color of the pavement is only slightly darker than that of untreated concrete.

CONCLUSIONS

Experience during the past five years in the laboratory and field show that Concrete Sealer, a coke oven coal tar material, definitely retards or prevents damage from ice removal chemicals to portland cement concrete used in highway and bridge deck construction. For best results it should be applied to new PC concrete pavements before opening to traffic. It has also been used to treat partially deteriorated pavement prior to resurfacing. It is advisable that it be used at intersections, bridge decks, interchanges, overpasses, steep grades, etc., where frequent use of ice control chemicals is expected.

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History and Application of Glass Fiber Reinforced Resin Wearing Surface

RAYMOND S. BLANCHARD, Northern Fibre-Deck Association

This paper discusses the concept and application of a new type of wearing surface. The material and its method of application were developed by members of the Northern Fibre-Deck Association in conjunction with the New York State Department of Public Works. This wearing surface was first used in New York State under a contract in which the Bureau of Public Roads participated.

The paper relates information regarding the concept and development of the wearing surface and the actual installation of a glass fiber reinforced resin wearing surface, and gives data regarding weight and cost of this type of application.

• A GREAT VARIETY of materials has been tested and used to increase the durability of concrete in bridge decks and wearing courses. This paper deals with the development and use of a glass fiber reinforced polyester resin as a laminate to provide a durable protective coating for these portland cement concrete surfaces.

In the search for a more efficient solution to the problem of achieving this protective coating for portland cement concrete it was proposed to use a reinforced plastic. Reinforced plastics have been used successfully in applications ranging from household utensils to heat shields for missiles. Particularly impressive is the demonstrated ability of a reinforced thermosetting plastic to act as a shield or impervious skin.

In adopting a program to utilize a thermosetting plastic in this field of concrete protection it became necessary to select from the vast field of plastics the basic resin and reinforcing material most suited to meet the structural and physical characteristics of a concrete bridge.

The resin chosen for this application was an isothalic polyester (manufactured by Selectron Division, Pittsburgh Plate Glass); the reinforcing material chosen was chopped glass fibers (manufactured by Fiberglass Division of Pittsburgh Plate Glass). These two materials were mixed together, and the necessary catalytic agents added immediately before application to the structural deck. Following this application, these materials underwent a chemical change or transition from a viscous liquid to a rigid solid. This transition was brought about by the catalytic agent which caused a copolymerization of the reactive monomer (in this instance, styrene) with an unsaturated polyester.

Problems were encountered in the development of mechanical apparatus with which to spray this viscous pre-mix of resin and glass. The original method used to apply these materials was a two-pot system. One side or pot of this system contained the premix with the addition of a promoter; the other side contained the pre-mix with a catalyst. These two mixtures were sprayed through separate units to blend by converging in the air before they came in contact with the deck.

The pre-mix was made by mixing together the ingredients for each side of the two-pot system separately. The mixture contained the resin; 3 percent by weight of ½-in. chopped strands of glass fibers; 1 lb per gal of 325 mesh silica flour as a thixotropic agent; and either the promoter, dimethylanaline, or the catalyst, benzoyl peroxide, depending on for which side of the system the particular mix was made. The polyester resin in this mixture had a specific gravity of 1.12 at 25 C and weighed 9.3 lb per gal.

Paper sponsored by Committee on Effect of Ice Control.

The glass fibers specified for use in this protective coating were a continuous roving pre-chopped into ½-in. lengths. These were added to reinforce the resin, as steel is added to reinforce concrete.

The ratio of glass to resin to form the pre-mix was predicated on the ability of the equipment to handle a glass and resin mixture rather than the optimum mixture to give ultimate results.

The pre-mix of polyester resin and glass was specified because anything less than the actual blending together of resin and glass fibers does not result in a complete wetting of each glass fiber. Fibers that have not become completely wetted with the resin could act as capillaries and transmit moisture through the laminate, thereby defeating the purpose of the application.

There are no data available to indicate that this type of material had ever been sprayed before. Consequently, there was not any commercial equipment available to handle this pre-mix. This required the engineering and construction of a mechanical spray unit to deliver this material. The original equipment designed to accomplish this task was a dual unit built in the following manner.

The pots or reservoirs were 55-gal steel drums with a cone welded inside to direct the gravitational flow of the material. These were mounted directly above positive displacement pumps, and the material was forced into a hose line and through a nozzle with a restricted orifice. Air was injected into the stream of the material as it arrived at the nozzle causing a vortex, and the resulting spray of materials from these two nozzles converged.

The glass fiber reinforced protective coating was first applied to the Brewerton Bridge in the fall of 1960. This bridge, actually two parallel structures, forms a link in the chain of the Federal Interstate Highway System known as Interstate 81 which crosses New York State and connects Pennsylvania with Canada. The Brewerton Bridge, sometimes referred to as the Oneida Lake Bridge, was at the time of its completion the longest span prestressed concrete bridge in the western hemisphere, measuring 460 ft between abutments with a clear center span of 320 ft 6 in.

One of the problems considered in the construction of this bridge was the protection of the thirty-four 240-ton, prestressed concrete girders from the deterioration caused by the reaction of de-icing chemicals with melting snow and ice on concrete.

It was for this project and future applications, calling for the protection of portland cement concrete, that this reinforced polyester resin protective coating was developed.

This system described for the application of this glass fiber reinforced protective coating, while mechanically sound, did not work because the catalyst in the pre-mix was activated by contact with copper or brass, and the glass strands would impinge on some internal projection and form a screen through which nothing but strained resin would pass.

Several changes in pumps were made during this first application to the Brewerton Bridge until a pump was found that did not have any objectionable metals (such as brass or copper) and at the same time would not lose output due to cavitation.

Changes were also being made in the mechanical delivery system. It was decided to remove, if possible, the catalyst from the pre-mix and inject it into the vortex in the nozzle. This would greatly simplify the equipment and permit the use of a one-pot system and at the same time increase the stability of the materials.

Using the two-pot system, the drums of catalyzed pre-mix had to be protected from sunlight and kept cool at all times to insure its pot-life. Otherwise, the catalyst would react and effect cure in the drum, thereby wasting the material.

A new system of catalyst injection was designed. This system was predicated on the fact that catalytic materials with lower viscosity were available. Such a material was methylethylketone peroxide which has a specific gravity of 1.120 at 25 C and a Brookfield viscosity of 12.7 centipoises at 25 C using a No. 1 spindle at 60 rpm as opposed to the benzoyl peroxide originally used which had a specific gravity of 1.2125 at 25 C and a Brookfield viscosity of 87,000 centipoises at 25 C using a No. 4 spindle at 6 rpm.

The manufacturer of the polyester resin agreed to the new approach and furnished a new polyester resin with cobalt naphthanate as the promoter and a thixotropic agent built into it.

With the successful development of material and equipment, supported by the techniques learned during the transition from idea to product, a new concept of concrete protection specified as "glass fiber reinforced resin waterproofing" was initiated.

Due to the method of constructing the Brewerton Bridge, the placing of 240-ton post-tensioned concrete girders necessitated the use of concrete filler strips between the girders. These filler strips had some depressions which had to be patched; this was accomplished by placing a patching mortar over an unclean surface. The patches were then steel troweled, though a steel troweled finish does not offer optimum conditions with which to form a bond.

The entire deck was swept clean and the glass fiber reinforced resin protective coating applied. Immediately after the application of the laminate and before a cure was accomplished, sand was hand sprinkled over the laminate to provide a mechanical key for the sheet asphalt wearing course which was to follow.

The application of the polyester laminate was completed in October 1960. It was then too late to apply the sheet asphalt wearing course in accordance with the New York State Department of Public Works specification which has a deadline of October 15. This resulted in the exposure of the polyester laminate throughout a severe winter. During this period of exposure the polyester laminate was subjected to the climatic conditions given in Table 1.

TABLE 1

CLIMATIC CONDITIONS FOR POLYESTER LAMINATE[a]

Date		Precipitation (days)	Snowfall (in.)	Number of Freeze-Thaw Cycles
Nov.	1960	19	2.7	12
Dec.	1960	26	27.8	9
Jan.	1961	24	37.8	14
Feb.	1961	17	25.8	13
March	1961	26	26.5	22
April	1961	26	8.4	11
Total		138	129.0	81

[a]From "New York Climatological Data," published by U.S. Department of Commerce.

Investigation of the glass fiber polyester resin laminate early in May 1961 showed several areas of random cracking and delamination from the concrete deck, prevalent in the locations where the deck had been patched.

The possible reasons for these failures were as follows:

1. Failure to employ surface preparation of the concrete deck other than sweeping. In all future applications, all areas to receive the polyester laminate should be either sandblasted or acid washed.

2. Failure of concrete patches to adhere to bridge deck over which the polyester laminate was applied. All patching of bridge decks to receive laminate should be avoided, and the depressed area brought to grade.

3. Cause due to uneven thickness of laminate because of inequalities of deck. As these materials develop exothermic heat during cure, a better control of the thickness of the one-pass spray application must be observed to eliminate too much heat that would cause residual stress that could cause cracking.

4. The difference in coefficient of linear expansion between reinforced concrete and polyester laminate. The resin used in this first application was an excellent quality,

but rigid, polyester that offered little to accommodate the linear expansion and contraction in reinforced concrete. Therefore, the type of polyester resin should be changed from a rigid to a resilient and the glass content increased to eliminate these differences in linear coefficient.

Despite the difficulties encountered, the application of these materials to the Brewerton Bridge can be termed a success. The areas of adhesive failure totaled less than 5 percent of the application. This represents an area of 2,150 sq ft out of more than 43,000 sq ft of protective coating material placed.

Since the completion of the repairs of the protective coating in spring 1961, the glass fiber reinforced resin waterproofing has undergone three specification changes and has been used extensively in New York State. The following is the current New York State Department of Public Works specification covering the application of this protective coating:

ITEM 450C—Glass Fiber Reinforced Resin Coating

1. Work. Under this item the Contractor shall furnish and place by spraying, glass fiber reinforced resin coating at the location shown on the plans and in accordance with this specification.

2. Materials. The resin shall be polyester. The polyester shall be Selectron Resilient Resin No. 5196 as manufactured by Pittsburgh Plate Glass Company, Paint Division, No. 1 Gateway Center, Pittsburgh 22, Pa., or an approved equal. The catalyst shall be Lupersol DDM manufactured by the Lucidol Division of Wallace and Tiernan Corporation, 1740 Military Road, Buffalo 5, New York, or Cadox MDP manufactured by the Cadet Chemical Corporation, Burt, New York. The glass shall be HSI with 805 sizing manufactured by Owens-Corning Fiberglas Corporation, Toldeo 1, Ohio, or an approved equal.

The glass fiber together with any inert filler, and the polyester resin shall be premixed before spraying. The glass fiber filaments shall exhibit complete wetting in the pre-mix solution. The chemicals to activate the setting reaction of the polyester shall be added to this mixture at the time of spraying and shall be so mixed or metered to establish a uniform distribution through the mixture. The chemical composition of these materials shall be so controlled that the curing time after spraying shall be ½ hr to 1½ hr. The mixture shall contain not less than 10 percent of glass fibers by weight. The glass fiber strands shall be ¼ in. long.

3. Preparation of Surfaces. Concrete surfaces shall be uniform and free from depressions and projections. They shall be cleaned of any accumulation of dirt, foreign materials or water to the satisfaction of the Engineer. Steel surfaces shall be sandblasted to clean metal and shall be free of all dust or foreign materials to the satisfaction of the Engineer.

4. Application of Coating. The coating material shall be sprayed by gun under pressure to provide a thickness of ⅛ in. minimum and ¼ in. maximum. Hand rolling shall be required to smooth out the surface irregularities resulting from the method of application. All rolling shall be done immediately after application before the surface cure is accomplished. Sharp, clean, dry sand passing a No. 8 sieve with 90 percent retained on a No. 16 sieve shall be applied before surface cure is accomplished so as to provide proper embedment and bond. The sand shall be applied at the rate of 3 lb per sq yd of surface.

This coating after application must possess excellent adhesion to the surface, evidence of which shall be obtained by tapping lightly with a ball-peen hammer 24 hr after complete cure has been accomplished.

After placing, the coating shall be protected in a suitable manner to insure against loads of any nature or damage by the elements until final cure has been accomplished or 24 hours' time has elapsed whichever is greater.

5. Approved. The use of "approved" in this specification shall require the affirmative action of the Deputy Chief Engineer (Bridges) in writing for the specific material, method or conditions to which it is applied.

6. Method of Measurement. The quantity of glass fiber reinforced resin coating to be paid for under this item will be the number of square feet of surface covered in ac-

TABLE 2

PHYSICAL PROPERTIES OF LAMINATE[a]

Property	Value
Brookfield viscosity, 25 C, poise	3.8
Specific gravity, 25 C, cured	1.24
24-hour water absorption (%)	0.12
Flexural strength (× 10^3 psi)	12.6
Flexural modulus (× 10^6 psi)	0.44
Tensile strength (× 10^3 psi)	8.9
Elongation at break (%)	1.19
Compressive strength (× 10^3 psi)	17.5
Barcol hardness	37

[a] Compacted from data sheets of three of the approved resin manufacturers.

cordance with the specifications as indicated on the plans or as ordered by the Engineer.

7. Basis of Payment. The price bid per square foot shall include the cost of furnishing all labor, materials and equipment necessary to prepare the surface, place the coating, apply the sand and complete the work.

These materials as specified have formed a part of 21 separate contracts involving the application of the polyester laminate to more than 70 separate bridges exceeding a total usage of 600,-000 sq ft.

The product used under today's specification contains a resilient isothalic polyester resin (manufactured by Selectron Division, Pittsburgh Plate Glass; American Cyanamid; Durez Plastics; and Molded Fiber Glass) and 10 percent by weight of HSI fiberglas with an 805 sizing (manufactured by Owens-Corning) to produce a laminate with the physical characteristics given in Table 2.

In many applications this laminate has been subjected to traffic from heavy road-building equipment without apparent injury. In one observed instance, a bituminous spreading machine had crushed a No. 3 size aggregate between its tread and the laminate without leaving a trace of any area of stress. This demonstrated ability of the laminate to withstand sudden impact without damage led to the belief that it could be used as a wearing course.

A testing program was then initiated under laboratory conditions to determine the validity of this premise.

During this program, tests were carried out to determine what aggregate would be most suitable to use to make the surface of the application skid resistant. Materials ranging from fine granules of iron ore tailings to particles of carborundem and aluminum oxide large enough to pass a No. 4 sieve were tried in varying ratios of resin to aggregate. One of the facts pointed out during this laboratory research program was that, although it was possible to form a homogeneous mixture of resin, aggregate and fiber glass, this mixture could not be placed with the spray equipment in one application. This fact required that if a material of this nature was to be applied as a wearing surface, it would have to be done in two successive spray coats to form a dual laminate. The first coat would be the application of the polyester materials as presently used. The second coat or wearing surface would be an external mixture of aggregate and resin applied directly to the first coat.

The aggregate decided upon was a crystallized silica having a gradation of an 8 to 16 sieve with a hardness factor of No. 7 on the original Moh scale.

On May 10, 1961, following 27 successive days of rain, this trial application was applied to a section of slab on grade known as Transit Road, located northeast of Buffalo, N.Y., in the suburb of Williamsville.

This trial application was to ascertain if the laminate could be made to perform two functions: that of a wearing course as well as its intended use as a protective coating for portland cement concrete.

In this trial application to Transit Road an area approximately 24 by 20 ft was chosen. This area ran from curb to mall across two lanes of pavement. This area was sand-blasted to remove dirt or other impurities present on the surface of the concrete and swept clean.

It was proposed to use a resilient polyester resin reinforced with 10 percent glass fiber as the first coat and on cure to apply a second coat or a rigid polyester resin filled with 60 percent crystallized silica as the wearing course. In performing this test an error was made, and the rigid and resilient resins inadvertently transposed. Therefore, in actual spraying the first coat was applied using the rigid resin; and the second coat, or wearing course, was applied using the resilient resin. Being aware of this error the test was completed, and this test section of roadway opened to traffic.

During the 6 months that this test area was subject to better than three million traffic vehicles, the laminate separated from the concrete highway in some sections along the trailing edge of the coating. It was concluded that this delamination was due to two reasons. First, because a polyester resin is water sensitive, it tends to cure without achieving its bond when the surface to which it is applied is damp. In this case, there was a very high water table present. Second, the rigid resin does not provide the same degree of adhesion as the resilient and therefore did not have the elastic properties necessary to accommodate the coefficient of expansion. Further, this test was performed on a slab on grade, which is not the location specified for the use of the end product. The test application, despite this delamination, was judged a success.

The coating was highly skid resistant and showed little wear from the passage of traffic. It also provided information that a coating of this type need not employ any but a resilient resin, as this resin held the crystallized silica intact.

In the July 1962 letting, a contract was awarded by the New York State Department of Public Works for the application of a glass fiber reinforced resin wearing surface for a test application on a structure.

The structures selected to receive this experimental wearing course were two three-span bridges with cantilever and suspended center span measuring 197 ft in length and 54 ft from curb to curb. These structures employ a 4-in. reinforced concrete paving slab. They are located west of Albany, N.Y., on Interstate 87, where it crosses over NY 5. They were completed and opened to traffic in October 1959.

In the preparation of the bridge decks to receive the application of the glass fiber reinforced resin wearing surface it was necessary to remove the existing joint material from the expansion joints and to chip out and clean the structural cracks.

The approach pavement and the existing bridge pavement being at the same grade necessitated the removal of a portion of the bridge pavement at the leading and trailing ends of the structure to prevent a bump. This was done by bush-hammering the slab down ⅛ in. and back approximately 6 in. The decks were then cleaned by means of sandblasting with garnet and wollastonite to remove any dirt, laitance, or other impurities present on the concrete surface to which the laminate was to be applied. The rate of cleaning the concrete decks by this method was approximately 5,000 sq ft per day. On completion of the sandblasting, the contaminations along with the expended garnet and wollastonite were then removed by power sweeper, then by vacuum cleaning, and finally by blowing with air. After the decks had been thoroughly cleaned, the cracks in the concrete were sealed with a polysulfide crack sealer.

On cure of the crack sealant, the first spray application of the protective coat of polyester resin reinforced with glass fibers was begun. This spray application was made to the entire deck with the exception of the expansion joints which were protected by boards wrapped with polyethylene. The spray application was applied over all longitudinal or construction joints.

In the application of a polyester resin, cleanliness and dryness of the surface to which these resins are being applied is essential. Therefore, extreme care was taken in this application to protect the deck from contamination. When rain occurred, the application was delayed until the surface was dry and clean before resuming application of the polyester resin laminate. On completion of each spray pass and before a cure had been accomplished, crystallized silica was hand applied to the laminate to help provide a mechanical key with the second coat or wearing course to be applied.

When the application of the first coat was completed and cured, the second course or wearing coat of polyester resin and crystallized silica was applied. This coating was likewise sprayed over the longitudinal joints while protecting the expansion joints.

The two coats form the glass fiber reinforced resin wearing course as applied under the following specification:

ITEM 460—Glass Fiber Reinforced Resin Wearing Surface

1. Work. Under this item the contractor shall furnish and place by spraying a two-

course glass fiber reinforced resin wearing surface at the location shown on the plans and in accordance with this specification.

2. (a) Materials. The first course resin shall be polyester. The first course polyester shall be Selectron Resilient Resin No. 5196 as manufactured by Pittsburgh Plate Glass Co., Paint Division, No. 1 Gateway Center, Pittsburgh 22, Pa. The catalyst shall be Lupersol DDM manufactured by the Lucidol Division of the Wallace & Tiernan Corporation, 1740 Military Road, Buffalo 5, New York, or Cadox MDP manufactured by the Cadet Chemical Corp., Burt, New York. The glass shall be chopped spun strand Type T-043, with a Chrome-Silane Hard sizing manufactured by Owens-Corning Fiberglas Corporation, Toledo 1, Ohio.

The glass fiber and the polyester resin shall be premixed before spraying. The glass fiber filaments shall exhibit complete wetting in the premix solution. The chemicals to activate the setting reaction of the polyester shall be added at the time of spraying and shall be added by meter and mixed so as to establish a uniform distribution throughout the mixture. The chemical composition of these materials shall be so controlled that the curing time after spraying shall be from ½ to 1½ hr. The mixture shall contain not less than 10 percent of glass fiber by weight. The glass fiber strands shall be ¼ in. long.

(b) Materials. The second course resin shall be polyester. The second course polyester shall be Selectron Resilient Resin No. 5196 manufactured by Pittsburgh Plate Glass Company, Paint Division, No. 1 Gateway Center, Pittsburgh 22, Pa. The catalyst shall be Lupersol DDM manufactured by the Lucidol Division of Wallace and Tiernan Corp., 1740 Military Road, Buffalo 5, New York, or Cadox MDP manufactured by the Cadet Chemical Corp., Burt, New York.

The chemicals to activate the setting reaction of the polyester shall be added at the time of spraying and shall be added by meter and mixed so as to establish a uniform distribution through the mixture. The chemical composition of these materials shall be so controlled that the curing time after spraying shall be from ½ hr to 1½ hr.

Aggregate in the amount of 60 percent by weight of the total mix shall be added at the time of spraying the second course. The aggregate shall consist of crystallized silica in the form of quartz having a hardness of No. 7 on the original Moh scale and a minimum specific gravity of 2.65 as produced by Industrial Silica Co., Youngstown, Ohio. The aggregate shall be graded to pass a No. 8 sieve with 90 percent retained on a No. 16 sieve.

3. Preparation of Surfaces. All surfaces shall be thoroughly cleaned of any grease, oil, tar, dirt, dust, paint and rust to provide a satisfactory bond between the contact surfaces and the applied materials. Any loose, scaled or damaged concrete shall be removed and the surface repaired to the satisfaction of the Engineer prior to application of the first course. The entire area to be coated with the wearing surface shall be cleaned by sandblasting. A light scrubbing with a 10 to 15 percent solution of muriatic acid followed by a thorough flushing with clean water shall be performed where required at the direction of the Engineer. The surface shall be thoroughly prepared, clean and dry, to the satifaction of the engineer before the first course is applied. The contractor shall adequately cover and protect painted metal surfaces adjacent to any sandblasted or acid-treated areas. Any damage to metal surfaces adjacent to areas cleaned shall be repaired and the surfaces restored to their original condition at the contractor's expense. The contractor shall also adequately shield the top of the existing premoulded bituminous joint material to prevent damage to this material during the sandblasting process.

The final stage of the cleaning shall consist of vacuum cleaning the entire surface. This cleaning shall be done at the time the wearing surface is applied. The vacuum cleaning equipment shall be the Tornado Series 400 Vacuum Cleaner Drum Conversion Unit.

4. Application of Wearing Surface. The surface shall be vacuum cleaned at the time the first course is applied. The vacuum cleaning equipment shall be operated ahead of the spray equipment by a distance not to exceed 20 ft. The first course resin shall be sprayed by gun under pressure to provide a minimum thickness of ⅛ in. Hand rolling shall be required to smooth out the surface irregularities resulting from the method of application. Rolling shall be done immediately after application before the surface cure

is accomplished. This course must possess excellent adhesion to the surfaces. After placing, the first course shall be protected in a suitable manner against loads of any nature or damage by the elements until the second course is applied. The first course shall be allowed to cure for a minimum period of 24 hr before application of the second coat.

(b) The surface shall be vacuum cleaned at the time the second course is applied. The vacuum cleaning equipment shall be operated ahead of the spray equipment by a distance not to exceed 20 ft. The second course containing the aggregate shall be sprayed by gun under pressure. Mixing shall occur at the special multi-headed nozzle under pressure and the coating shall be sprayed on the surface of the first course to a thickness of $\frac{1}{4}$ in. $\pm$ $\frac{1}{16}$ in. This application must possess excellent adhesion to the surface. After placement, the second course of the wearing surface shall be protected in a suitable manner to insure against loads of any nature or damage by the elements until final cure has been accomplished or a minimum period of 24 hr.

5. Approved. The use of "approved" in this specification shall require the affirmative action of the Deputy Chief Engineer (Bridges) in writing for the specific material, method or conditions to which it is applied.

6. Method of Measurement. The quantity of glass fiber reinforced resin wearing surface to be paid for under this item will be the number of square feet of surface covered and acceptable in accordance with these specifications and where indicated on the plans or as ordered by the Engineer.

7. Basis of Payment. The price bid per square foot shall include the cost of furnishing all labor, materials, and equipment necessary to prepare and clean the surfaces, place the two course wearing surface and complete the work.

On completion of the application of the glass fiber reinforced resin wearing course, the bridges were reopened to traffic. The southbound structure was reopened on August 20, 1962, and the northbound bridge on September 4, 1962.

The traffic on these two structures is 23,933 vehicles per day. The greatest volume of traffic occurs between the hours of 4:30 and 5:30 PM, at which time the traffic on the southbound bridge represents approximately 46 percent of the total flow during this period. This is counted at 1,100 vehicles per hour. The traffic over the northbound bridge is counted at 1,348 vehicles, during this same period of time.

This glass fiber reinforced resin wearing course, when applied in accordance with the specifications above, has a thickness of $\frac{3}{8}$ in. and weighs approximately 30 lb per sq yd. This is only $\frac{1}{10}$ as heavy as a 3-in. wearing course of bituminous macadam.

This glass fiber reinforced resin wearing course, though exposed to traffic for a limited time, is providing the concrete protection for which it was designed. This laminate is resistant to hydrocarbon solvents, it is impervious to the penetration of moisture, it is highly skid resistant, it is not affected by de-icing chemicals, and there has been no evidence of failure due to cracking or delamination.

A Concrete Bridge Deck Survey by the SUR/FAX Photographic Method

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As part of a study of the deterioration of concrete bridge decks, a condition survey of 18 structures was made with SUR/FAX photographic equipment, developed as a tool for conducting surveys of pavements. The purpose of the study was to determine the ability of the photographic equipment to detect typical defects in concrete bridge decks. The structures surveyed, 15 of which were less than five years old, were selected to include crazing, disintegration, hair checking, map cracking, pitting, popouts, surface scaling, and transverse structural cracks.

The results of the photographic survey were compared with those of a detailed visual survey. The general conclusion drawn was that additional development was needed in the SUR/FAX method to make it practical for general use in surveys of bridge decks.

• IN RECENT YEARS, the problems associated with deterioration of portland cement concrete bridge floors have received increased attention. Particular emphasis has been placed on the effects of de-icing chemicals on this deterioration. Determination of the contribution of these agents to the deterioration has been given a high priority in the recently initiated National Cooperative Highway Research Program. Extensive studies and condition surveys have been conducted and reported (1).

Condition surveys such as those required for field observations of structures are time consuming and expensive; in many cases, the results are of a subjective nature difficult to express quantitatively. The need exists for a method of survey that will be rapid and will make data available in a form more practical for office or laboratory evaluation.

A method has recently been developed for making pavement condition surveys by photographing the pavement on a continuous film similar to that used in aerial photography. This method, designated SUR/FAX, offers the advantages of speed, permanent recording, and, if used extensively, relatively low cost (2).

The method has been studied experimentally and found to have certain promise, particularly in the surveying of portland cement concrete pavements, in which photographic contrast is high (3, 4). The possibility of applying the method to portland cement concrete bridge decks formed the basis of the study reported in this paper. It was recognized that a number of practical problems would limit the use of the method for surveying bridge decks exclusively. The relatively great distances between structures, for example, would result in an inefficient operation and thus increased cost. On the other hand, if the method did prove practical for the routine surveying of pavements, a number of structures would be covered in the process, and the extent to which the method as generally applied would be successful in determining bridge defects would

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be important. The fact that the method would successfully detect defects in portland cement concrete pavements does not assure that it would be of value for bridge deck surveys because, in the latter case, small and less apparent defects would be of structural significance. It should be emphasized then that the SUR/FAX method as studied in this investigation was employed in a situation requiring resolution and magnification greater than was necessarily anticipated by the manufacturer in developing it for pavement surveys, but the normal procedures were employed because they would be the ones most likely to be extensively used.

CRITERIA

Most of the defects encountered in the study were typical of those found in portland cement concrete pavements but in many cases the degree of distress was less than that normally associated with pavements. This would be particularly true of the defects listed later as crazing, transverse cracking, and map cracking. A considerable effort was expended in trying to determine the ability of the equipment to detect very fine map cracking because of its applicability to another study under way (5). It was recognized that this phase of the testing was severe; therefore, it was given minimal significance in the over-all evaluation.

In this study, all defects were classified in accordance with the terminology used in HRB Special Report 30. From this report, under the heading of Defects and Manifestations, the following terms and definitions are presented to describe the types of distress:

> Crazing—Pattern cracking extending through only the surface layer; a result of more drying shrinkage in the surface than in the interior of the plastic concrete.
>
> Disintegration—Deterioration into small fragments or particles due to any cause.
>
> Hair checking—Small cracks not conforming to a regular pattern which extend to an appreciable depth but not to the full depth of the pavement; occurring before the concrete takes its final set.
>
> Map Cracking—A form of disintegration in which cracking of the slab surface develops in random pattern resembling the political subdivisions on a map.
>
> Pitting—The displacement of individual particles of aggregates from the pavement surface, without major displacement of the cementing material or mortar.
>
> Popouts—Craterlike depressions caused by the breaking away or forcing off a portion of the pavement surface by the expansion of a piece of underlying coarse aggregate.
>
> Scaling (Surface)—The peeling away of the surface mortar of portland cement concrete exposing sound concrete even though the scale extends into the mortar surrounding the coarse aggregate.
>
> Transverse Cracking—Approximately vertical cleavage due to natural causes or traffic action which follows a course approximately at right angles to the centerline.

To provide a means by which the effectiveness of the photographic method could be determined, all bridge decks in the study were visually surveyed and all defects identified and plotted to scale. For this study, pitting and surface scaling were considered sufficiently similar to be grouped as one defect type.

In the detailed sketches, crack widths were recorded as: <0.005 in., 0.005 to 0.010 in., 0.010 to 0.025 in., 0.025 to 0.050 in., 0.050 to 0.100 in., 0.100 to 0.125 in., 0.125 to 0.250 in., and >0.250 in. Areas of distress, such as disintegration, and pitting and surface scaling, were approximated to the nearest square foot.

EQUIPMENT

The photographic apparatus used in this study consists principally of a 35-mm camera mounted atop a retractable hydraulic mast. A panel body truck is used to transport the camera-mast assembly (Fig. 1).

During the photographing operation, the camera assembly is raised approximately 14 ft, and when traveling between sites it is lowered to about 10 ft. To lessen the possibility of any relative movement between the camera and the surface being photographed, the camera is tilted upward to a rectangular mirror which reflects the pavement image through the shutterless slit of the camera onto a continuously moving film. Shock absorbers and stabilizer arms are attached to the mast to protect the camera assembly against shock and to prevent it from moving while in the photographing position. Banks of lights directed downward at an angle of approximately 30° from the

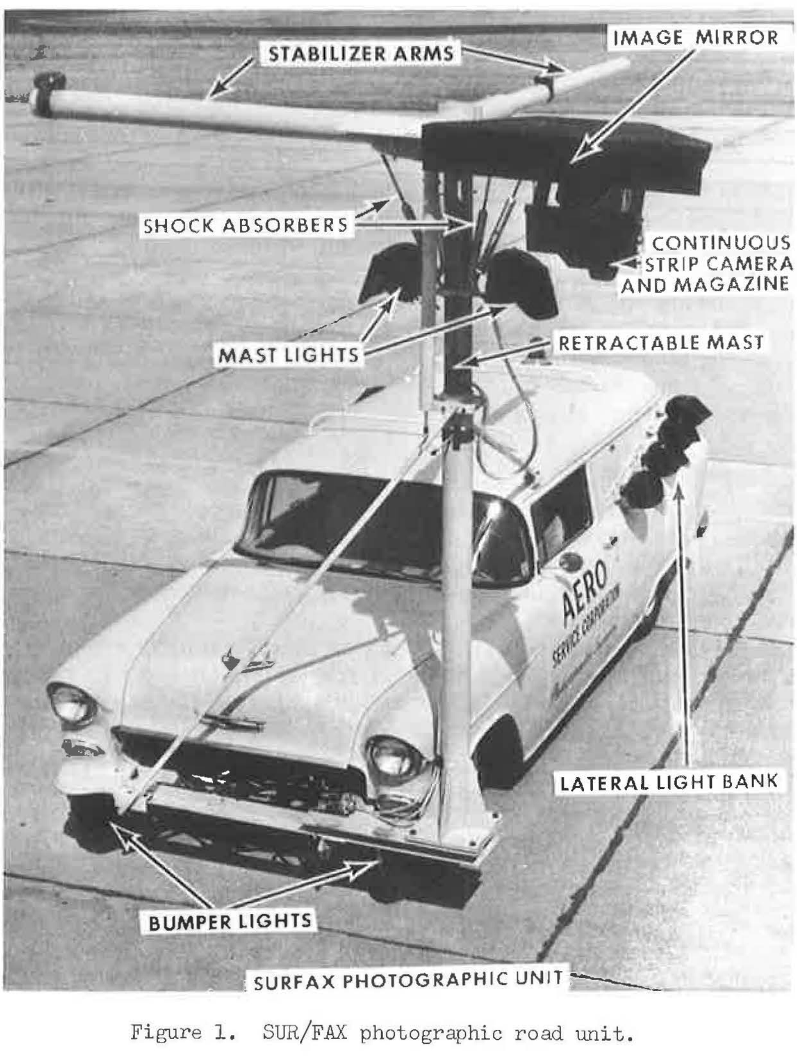

Figure 1. SUR/FAX photographic road unit.

horizontal are mounted on the truck sides, below the front bumper, and on the mast to permit photographing to be done at night. The use of artificial lighting serves a threefold purpose by permitting greater control over the light intensity, reducing the effects of shadows, and allowing the photographing to be accomplished during the hours of minimum traffic. A generator carried in the rear of the vehicle furnishes current for the lighting system.

In the photographic operation, as used in normal pavement surveys, the truck travels at 25 to 30 mph in the right-hand lane photographing both the right- and left-hand lanes simultaneously. Between sites, with the camera-mast assembly in the lowered position, speeds of 45 to 50 mph are commonplace. In this study, the photographic operation was generally conducted by two-men teams provided by the Aero Service Corporation.

PROCEDURES

Two photographic surveys were made for this study. In the original survey, 18 structures were photographed with the same equipment and procedures as are used in the normal pavement surveys. On the basis of the results of this original survey, it was mutually agreed that rephotographing with modified procedures and equipment should be attempted. In the modified survey, the five structures exhibiting the most severe cases of distress as shown by the original survey were rephotographed with the same equipment, except that improved lens and lighting systems were used.

The camera used in the original survey was equipped with a wide angle lens which enabled pavement widths up to 36 ft to be photographed in a single traverse. In the modified survey, after the sensitivity of the camera's lens system was improved, the width of pavement encompassed was reduced to approximately 20 ft. The degree of resolution obtained in the modified survey was increased considerably over that of the original survey by these modifications; however, most bridge decks in Virginia are 24 ft or greater in width, so two traverses of the structure were required to photograph the entire deck surface. As an aid in checking the degree of resolution obtained in the photographs, 2-ft square resolution charts similar to those used in eye examinations were placed on the decks prior to photographing.

From the exposed film, the Aero Service Corporation prepared and furnished enlarged glossy and matte prints, and 35-mm film positives for study purposes. The enlargement of the glossy and matte prints resulted in an average scale of 1 in. equals 6 ft. However, the results and conclusions of the study are based solely on the data obtained from the enlarged glossy prints.

RESULTS

In selecting the structures for photographing, considerable attention was given to both the extent and the severity of the distressed area so as to include as large a range of conditions as possible. Of the 18 bridges photographed in the original survey, 5 were selected as being most representative of the group, and these were rephotographed in the modified survey.

The glossy and matte prints and film positives furnished for each bridge were compared, the film positives being projected on a screen and viewed with the aid of a variable speed projector. On the basis of the comparisons it was concluded that greater detail could be detected from the glossy prints than from the other two means. Hence, the data and conclusions of this study were obtained entirely from the information provided by the enlarged glossy prints.

In the original survey there were occasions when the prints of the wider decks contained both unusually dark and unusually light areas, resulting in a decrease in the degree of resolution. This was believed to be caused by variations in light intensity. In the modified survey, additional lights were mounted beneath the front bumper, and a narrower width of roadway was photographed. As a result, a decrease in the number of dark and light areas was noted.

In both photographic surveys there were instances in which the longitudinal scale of the prints was not constant for a given bridge deck. The distortion in these cases was as great as 3 ft in a 50-ft slab. In the modified survey the number of instances of distortion in the longitudinal direction was reduced, but not eliminated, through the addition of a fifth wheel arrangement to synchronize the camera and vehicle speeds. Further, within each photographic survey the quality of the prints was different in that the clarity and detail were better in some cases than in others. The prints from both photographic surveys were compared with the sketches from the visual survey, and, because the photographs from the first survey were of poorer quality than those of the second, it was believed desirable to base the conclusions of this study on the results of the modified survey only. To point out the increase in percent of defects detected in the modified survey over that of the original survey, data from both surveys are shown in Figure 2. These data were obtained by comparing the photographs with the detailed sketches of the bridge decks. In the detailed sketches unusual effort and precision were employed in the attempt to obtain a true description of the deck condition so that a fair and complete evaluation of the photographs could be obtained. In these sketches the extent of the distressed area and each visible crack were sketched to scale and classified according to the HRB system. In most cases an average was taken of two or more persons' work in determining the number of defects found from the photographs.

The results of the visual and modified surveys are given in Table 1.

Column 2 gives the number of that particular defect detected from sketches of the visual survey; column 3 gives the number of these same defects detected from the glossy prints; and column 4 gives the percent of defects present which were detected from the prints. Column 4 shows that the data fall into two broad classes. The defects in Class A (crazing, hair checking, and map cracking) are the more difficult to detect. Class B includes the defects that were more easily detected; such as disintegration, pitting and surface scaling, popouts, and transverse cracks.

The defects in Class A were more difficult to detect because of the relatively small width of cracks occurring in these types of distress. Of the bridge decks observed in this study it was found that the crack openings of the defects in Class

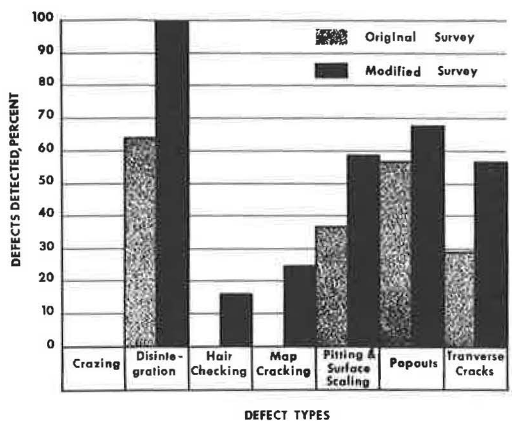

Figure 2. Percent defects detected in surveys.

TABLE 1

COMPARISON OF DEFECTS DETECTED FROM SKETCHES AND PRINTS

Type of Defect	Number of Defects Detected		Percent Defects Visible from Prints
	Sketches	Prints	
(1)	(2)	(3)	(4)
Crazing	4	0	0
Disintegration	11	11	100
Hair checking	497	78	16
Map cracking	20	5	25
Pitting and surface scaling	242	142	59
Popouts	44	30	68
Transverse cracks	7	4	57

A ranged from 0.01 to 0.25 in. Table 1 shows that the SUR/FAX equipment was generally unsuccessful in detecting cracks in this width range. Only in instances where the area of distress and crack openings were comparatively large could the defects be noted from the prints. Occasionally the larger areas of these defects were recognized from the photographs as an inconsistency in the general appearance of the surface. However, if a distressed area was noted, it was generally impossible to identify the particular type of distress. Furthermore, many times the distressed area could not be noted because it was not sufficiently large to offer adequate contrast to the surrounding unaffected area. Under the conditions of this study it was concluded that distressed areas containing cracks of widths less than 0.25 in. could not be consistently detected nor could they be classified on detection from the photographs.

A greater percentage of the defects in Class B were noted from the photographs for the following reasons:

1. In the case of disintegration and of pitting and surface scaling a very noticeable contrast exists on the prints between these areas of distress and the unaffected areas. In addition to the aid in detection offered by the contrasting surface appearance, the detection of these types of defects was also increased by their depth, which caused pronounced shadows to appear on the surface of the deck. Because the system of lights was directed on the deck surface from an angle, the small vertical face of the defect farthest from the camera would reflect a relatively large amount of light to the camera system. Consequently, the nearer face of the defect was hidden to the camera and hence no light was reflected. This resulted in light and dark areas appearing on the prints and served to outline the extent of the distressed area.
2. In a similar manner, the shadow effects resulting from popouts and transverse crack openings larger than 0.25 in. appeared on the prints as black dots and dark lines, respectively. In the case of crack detection, it was found that the length of cracks is an important factor in that for cracks of equal width the crack with the greater length was more easily noted.

In a supplemental experiment in which the deck surface of a bridge containing map cracking was sprayed with water, it was observed that the detection of the cracks was considerably easier if the surface was photographed at various stages of the drying-out process. When a concrete surface containing cracks is wetted sufficiently and then allowed to undergo drying, moisture remains in the cracks for a longer period than it does on the surface. This condition, in which the dampened cracks were of darker color than that of the dry surface, accentuated the contrast between the unaffected and distressed surfaces as they appeared on the photographs. (Possibly, this contrast could be enhanced through the use of a wetting agent.) As a result, it was found that the detection of cracks in a distressed surface is increased substantially when the surface is photographed in a state of differential drying.

In a further effort to increase the detection of cracks by the SUR/FAX method, infrared film was used to photograph the bridge deck sprayed with water. As before, the photographing was conducted when the surface was in a drying-out process; however, the results of this experiment proved to be of less value than those received from the conventional method.

CONCLUSIONS

As a result of this work the following conclusions appear justified:

1. The SUR/FAX methods and equipment used in the original survey, and representative of the techniques normally employed in pavement surveys, are not suitable for detection of many of the types of distress common to bridge decks.
2. The particular procedures used in the modified survey were better adapted to the envisioned concepts of this study; however, additional development is needed in the SUR/FAX method to make it practical for general use in surveys of bridge decks.
3. The results obtained from the procedures used in the modified survey indicate the following:

(a) The detection of defects was greater from the enlarged glossy prints than from the matte prints or the projections of the film positives.
(b) Crack openings less than 0. 25 in. cannot be consistently detected. Neither can crazing, hair checking, or map cracking be distinguished from each other at crack openings less than 0. 25 in.
(c) Types of distress exhibiting a depth dimension that reflects light are more easily noted.
(d) Longer cracks are more easily detected than shorter cracks of the same width.

4. The detection of map cracking and similar types of distress is greatly enhanced from photographs of the distressed surface undergoing differential drying.

ACKNOWLEDGMENTS

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Polyester Overlays for Portland Cement Concrete Surfaces

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The use of thermosetting polyester resins in protective coatings for portland cement concrete (PCC) surfaces is described. Specific reference is made to the use of polyester-aggregate systems as coatings on PCC bridge decks.

Two polyester overlay systems are examined. One uses ordinary seal coat application methods in which polyester resin is sprayed directly onto a carefully cleaned PCC surface. Stone chips are added for skid resistance. The second system uses a polyester-sand mortar applied as a ¼-in. overlay. This type of system also acts as a leveling course which allows readjustment of irregular wearing surfaces to a desired grade.

Selective gradation of the aggregate and proper proportioning of the graded aggregate with polyester resin give a dense impermeable mortar overlay. The variation of compressive strength with aggregate grading is demonstrated.

The properties of selected polyester mortars are examined in the laboratory. The mortar is shown to reach 80 percent of its ultimate compressive strength in less than 24 hr. The completed mortar is also shown to be resistant to hydrocarbon solvents.

Methods used for surface preparation and for applying the polyester overlays are examined. Special construction equipment is described. Both overlay systems have been shown to be effective in several large-scale field tests.

• MILLIONS of dollars are being spent annually for the maintenance and repair of badly spalled and deteriorated portland cement concrete (PCC) bridge decks. This problem is particularly acute in northern areas of the United States where the infiltration of brine, which results from the use of de-icing salts, together with a large number of freeze-thaw cycles are the major causes of rapid deterioration of concrete. Degradation also occurs on bridge decks subjected to vibration from heavy truck traffic. Often deterioration of wearing surfaces is severe enough to require removal of the PCC sections.

The New York State Thruway Authority launched in 1958 a 4-yr, $3,000,000 rehabilitation program to seal and waterproof over 300 of their bridges (1). Estimates based on private discussions indicated over 120,000,000 sq yd of PCC decks in the United States are in need of resurfacing. For example, 8,500 bridges need to be recoated in one southern State alone. Another 2,200 PCC decks require resurfacing in a midwestern State.

This paper describes two overlay systems using polyester resin. One method is based on ordinary seal coat application techniques. A second method involves the use of a polyester-sand mortar. Both the mortar and its method of application are claimed in patent applications now pending before the U. S. Patent Office.

These polyester materials provide a tough, impermeable, fuel-resistant coating of approximately ¼-in. thickness. The thermosetting nature of polyester resins allows

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the coatings to harden quickly and thus reduces considerably "out-of-service" time.

EXISTING OVERLAY SYSTEMS

One of the most common methods for resurfacing PCC bridge decks is with 1 to 2 in. of asphalt concrete (AC). This method can be expensive when the cost of raising existing expansion joints and gutters and changing drainage outlets and bridge approaches are considered. In addition, AC overlays increase the dead weight of the bridge. This increase alone is sometimes sufficient to preclude the use of AC as a resurfacing material for PCC bridges.

Recently, latex-modified cement (LMC) mortars have been introduced in several States as overlays for PCC decks (2). Despite improved compressive, tensile, flexural, and bond strengths claimed for LMC when applied as a ½-in. coating over ordinary PCC, State highway officials and research engineers have had only limited success with LMC (3). In particular, long curing times are required with the LMC mortars.

A third type of coating, epoxy road surfacings, was first introduced in 1954 (4, 5). Both skid-resistant seal coat applications and epoxy asphalt concrete (EAC) mixes were extensively tested in the laboratory and in the field (4, 6, 7, 8). Once correct methods for surface preparation and bonding of the overlays to pavements were developed, epoxy coatings proved quite successful (9). Epoxy resin, like polyester resin, is a thermosetting material that allows the rapid formation of a tough protective surface coating. These coatings have good chemical resistance, can withstand severe weather cycles, and are wear resistant. Of the two, polyester resin is less expensive and currently costs about one-half the price of most epoxies (10).

DEVELOPMENT OF POLYESTER SYSTEMS

Seal Coat

Ordinary seal coat methods were used in the early experiments with polyester resins as protective coatings (11). Aggregate chips were bonded to a cleaned PCC surface with polyester resin. Figure 1 (11) shows the condition of two seal coat sections after 2½ yr of heavily loaded truck traffic. Since the time of taking the photograph an addi-

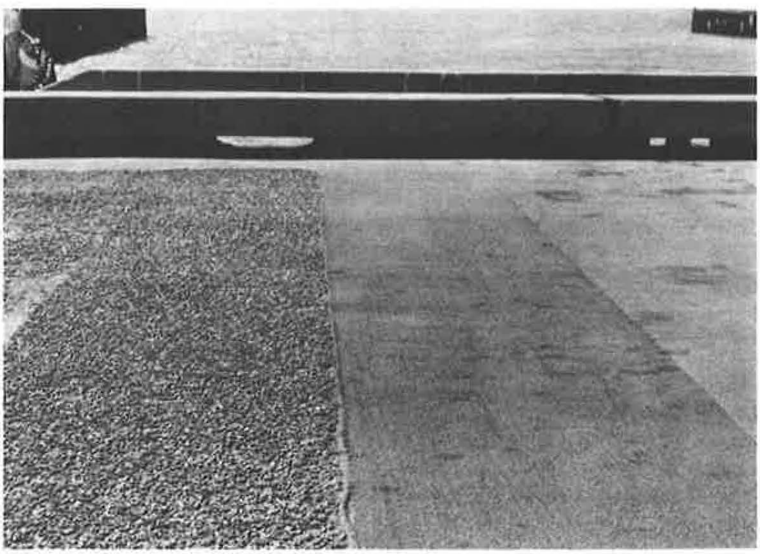

Figure 1. Condition after 2½ yr of service of large and small aggregate seal coats bonded directly to PCC with polyester resin.

tional year of traffic has passed over these sections with little sign of wear. This type of application is designed for new concrete surfaces or for relatively undamaged PCC decks where a protective coating is desired.

The seal coat is not acceptable for rough and deteriorated PCC surfaces which require grade correction. These decks need a coating that can "smooth out" the riding surfaces as well as protect the remaining sound concrete. Studies were carried out in the laboratories to develop a suitable polyester-aggregate mortar for this condition also.

Mortar Overlay

A promising mortar was field tested for the first time on a heavily traveled bridge in the Richmond Refinery of the Standard Oil Company of California. Initially, a high impact rigid polyester resin was used to prime the PCC deck prior to applying the ¼-in. mortar overlay. A high impact rigid polyester was also used as the binder in the mortar. Failures appeared in the form of cracks in the overlay and bond failures between the overlay and PCC surface. These failures were attributed to either a higher modulus of elasticity in the overlay than in the concrete base or the inability of the rigid resin to withstand stresses set up during the curing of the overlay or during thermal cycling.

In the next field trials, a more flexible, high impact polyester resin was substituted for the high impact rigid polyester in the primer and in the mortar. In addition, better surface preparatory methods and a more efficient prime coat system were used. The sections were placed during April 1962 on a smaller PCC bridge on the same access road as the first trials in the Richmond Refinery. Both sandblasting and acid etching were tried as surface preparatory methods. Although no significant difference was found from bond tests (4) on the acid-etched vs the sandblasted sections of the study area, acid etching has been shown to be more effective for epoxy surfacings (4). However, the type and condition of the concrete surface can significantly affect test results (4). After 9 months' service, there are no signs of cracking or bond failures. The condition of one of these sections is shown in Figures 2 and 3. These photographs were taken after 4 months' service, and since then there has been no difference in appearance. The rippled surface texture shown in Figure 2 was caused by the use of a polyethylene film in the compacting process. Apparently, slippage of the film when

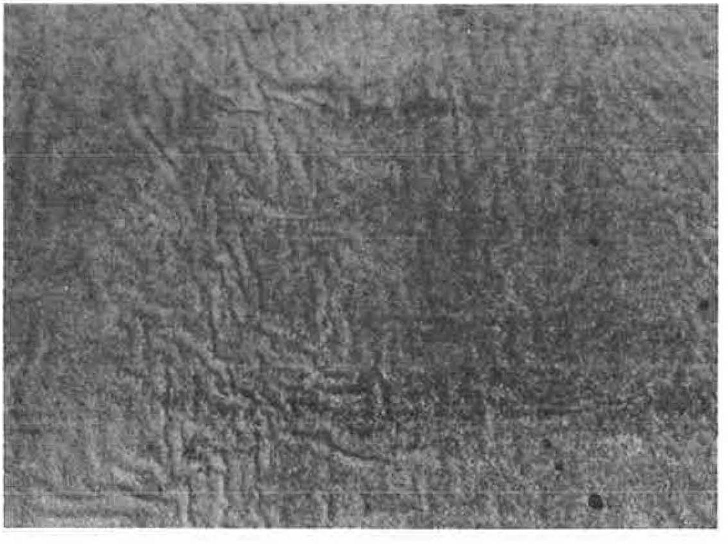

Figure 2. Polyester mortar overlay on PCC surface.

passed between the compacting roller and mortar is responsible for the rippled appearance in the mortar's surface. Placing techniques are discussed in more detail later in the paper. The core holes in Figure 3 are the result of in-place tensile tests which are also discussed later. Additional field trials are being conducted on publicly traveled bridges.

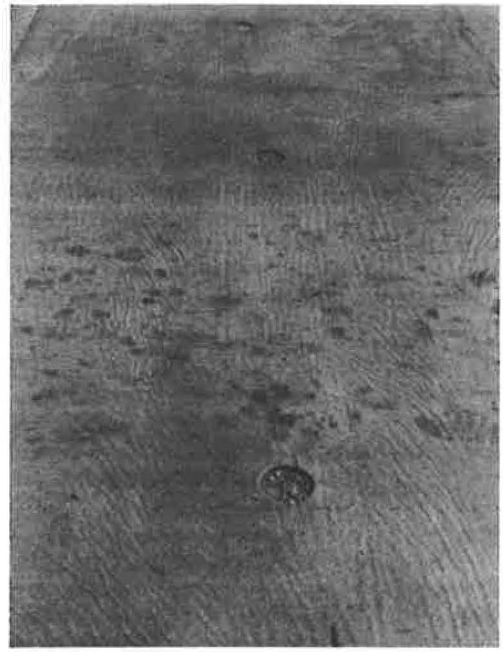

Figure 3. Condition of polyester mortar section after 4 months' service.

AGGREGATE GRADING IN MORTAR FORMULA

A critical range of aggregate gradation must be met to attain maximum strength in the mortar. Previously, Andreasen and Anderson (12) showed in their analyses of aggregate grading curves that they can be represented by

$$y = CK^q \qquad (1)$$

in which

- y = percent aggregate passing a sieve of size K;
- q = slope of plot of log percent passing each sieve vs log of sieve size; and
- C = a constant.

This equation was later plotted on a double-logarithmic scale by Wilhelmi (13).

Nijboer (14) subsequently found that the voids in mineral aggregate for use in AC mixes can be determined graphically. He showed that a slope q of the plot of log percent passing various sieves vs log of sieve size is 0.45 for an aggregate gradation having minimum voids or maximum density.

When thermosetting polyesters were considered as a possible component for the preparation of sand mortars for use in paving work, their particular nature made prediction of the properties of the ultimate cured overlay virtually impossible.

It was also found in an extensive series of tests that a superior mortar is obtained by selective grading of the aggregate according to

$$\log Y = a \log X + b, \qquad (2)$$

in which Y is the percent by weight of the aggregate that will pass through the maximum sieve opening of X inches, and a and b are constants ranging from about 0.35 to about 0.55 and from about 2.30 to about 2.65, respectively. By choosing these critical proportions of the so-graded aggregate and polyester resin, one obtains a mortar that will set to an overlay having greatly improved properties, in particular, the compressive strength, as compared with previously proposed PCC and AC mixes.

The relationship of the percent of aggregate passing various sieve sizes, shown as line B in Figure 4, was developed for a No. 8 mesh maximum size sand. During construction, it is often necessary to modify the preceding grading to improve the workability of the mix. Moreover, other maximum size aggregate mortars may be desired. For these reasons, lines A and C in Figure 4 are included as reasonable limits to the ideal grading.

Four sand fractions are used in the mortar formula. The grading of each fraction is given in Table 1. When combined as 40.6 parts coarse sand, 16.3 parts medium sand, 24.4 parts fine sand, and 18.7 parts filler, the grading of line B in Figure 4 is closely approximated.

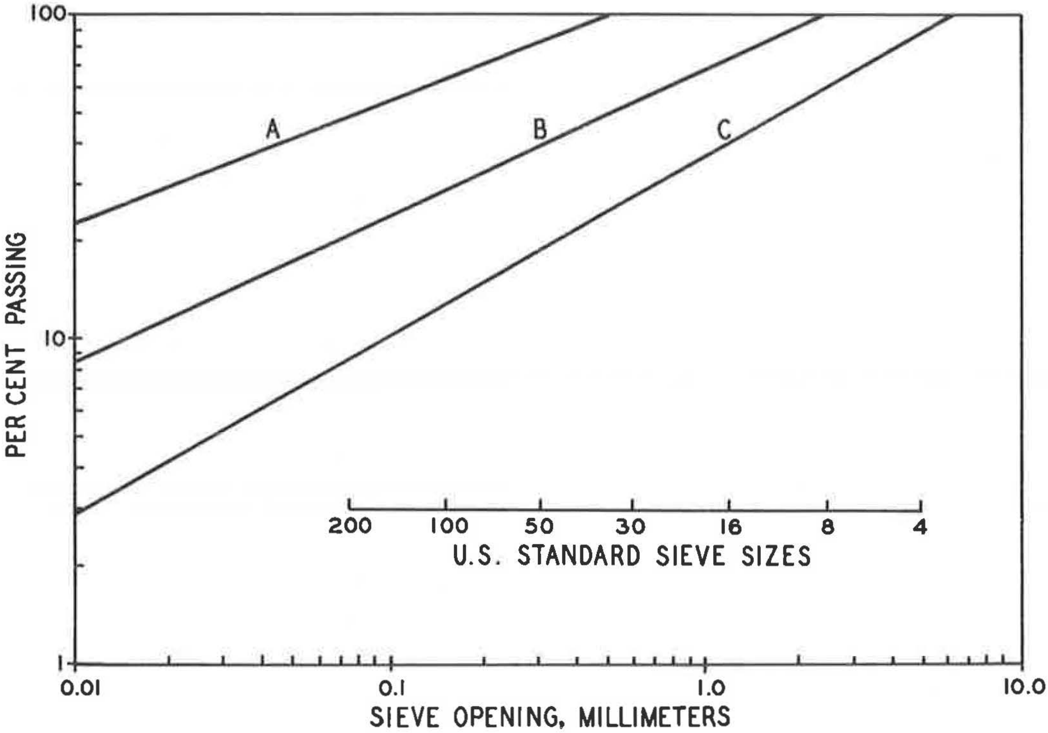

Figure 4. Aggregate grading curve and allowable limits for maximum compressive strength of polyester mortar.

TABLE 1

GRAIN SIZE DISTRIBUTION DATA FOR MORTAR OVERLAY AGGREGATE

ASTM Sieve Designation	Square Opening (mm)	Cumulative Percent Passing by Weight			
		Coarse Sand	Medium Sand	Fine Sand	Filler
No. 8	2.380	100.0			
No. 12	1.680	78.0	--		
No. 16	1.190	26.0	100.0		
No. 20	0.840	2.0	81.0		
No. 30	0.590	--	23.0	--	
No. 40	0.420		4.0	100.0	
No. 50	0.297		1.0	99.5	
No. 70	0.210		--	93.8	
No. 100	0.149			58.2	
No. 140	0.105			21.2	
No. 200	0.074			5.0	--
No. 270	0.053			1.3	100.0
--	0.040			--	99.0
--	0.030				94.0
--	0.020				73.0
--	0.010				39.0
--	0.005				18.0

The resin content to be used in the mortar is estimated graphically using the method previously discussed for determining the voids in the mineral aggregate. The resin content selected is equal to or slightly higher than the calculated voids so that the resultant mortar is voidless.

The importance of aggregate grading is further shown by the compressive strength values given in Table 2. Two-inch cube specimens prepared from the mortar were tested according to ASTM procedure C 306-60 (Method of Test for Compressive Strength of Resin-Type Chemical-Resistant Mortars (C 306-60), 1961 Book of ASTM Standards, Part 4, p. 447) with the exception that the speed of testing was 0.05 in. per min, and an accelerated cure was used. The curing periods were 140 F for 1 hr and 325 F for 3 hr. The samples were then allowed to cool to room temperature for 1.5 hr before testing. The polyester used in these samples is of the high impact rigid type. A high impact flexible polyester mortar would give somewhat lower compressive strengths due to the longer time required for the flexible resin to reach its ultimate strength.

LABORATORY PROPERTIES OF POLYESTER MORTAR

The mortar formula found to be the most satisfactory from the laboratory and field studies was examined for compressive strength, modulus of elasticity, tensile strength, air permeability, and solvent resistance. The sand proportions and resin concentration used in this mortar are given in Table 3. The higher resin concentration shown in Table 3 vs Table 2 is due to the difference in specific gravities of high impact rigid and high impact flexible polyester resins. Results of physical tests are given in Tables 4 and 5.

TABLE 2

AGGREGATE GRADING INFLUENCES COMPRESSIVE STRENGTH OF POLYESTER MORTAR

Proportions (parts per hundred)					Compressive Strength (psi)
Coarse Sand	Medium Sand	Fine Sand	Filler	High Impact Rigid Polyester Resin	
90.9	0	0	0	9.1	5,150
72.7	18.2	0	0	9.1	5,270
45.5	45.4	0	0	9.1	5,570
27.3	63.6	0	0	9.1	4,750
0	90.9	0	0	9.1	4,900
63.6	0	27.3	0	9.1	8,650
27.3	36.3	27.3	0	9.1	7,780
27.3	18.2	45.4	0	9.1	5,400
54.5	18.2	18.2	0	9.1	7,200
45.4	27.3	18.2	0	9.1	8,720
54.5	9.1	27.3	0	9.1	8,750
45.4	18.2	27.3	0	9.1	9,040
36.3	27.3	27.3	0	9.1	8,600
54.5	0	36.4	0	9.1	8,350
36.3	18.3	36.3	0	9.1	8,190
44.5	17.8	26.8	1.8	9.1	8,470
37.6	15.0	22.6	15.7	9.1	10,070
36.9	14.8	22.2	17.0	9.1	10,930[a]

[a]Aggregate grading corresponds to line B in Fig. 4.

The compressive strength and modulus of elasticity values represent an average of three tests on 2-in. cube samples made according to ASTM procedure C 306-60 with slight modifications in curing methods and testing speed. The mortar's tensile strength is an average of four results on samples prepared by ASTM procedure C 307-61 (Method of Test for Tensile Strength of Resin-Type, Chemical-Resistant Mortars (C 307-61), 1961 Book of ASTM Standards, Part 4, p. 452) with similar modifications. Both compression and tension samples were cured for 24 hr at room temperature (70 F) before testing. The testing speed in both instances was 0.05 in. per min. The air flow rate through 2-in. diameter by ¼-in. thick mortar plugs recovered from one of the field trials was measured in the laboratory with the air permeability apparatus (17). The retention of strength after immersion in various hydrocarbon solvents was measured on samples air cured at ambient temperature for 24 hr before the soaking period began.

TABLE 3

COMPOSITION OF POLYESTER MORTAR OVERLAY

Material	Proportions (parts per hundred)
Coarse sand	37.5
Medium sand	15.0
Fine sand	22.5
Filler	15.0
High impact flexible polyester resin	10.0

TABLE 4

PHYSICAL PROPERTIES OF POLYESTER MORTAR COMPARED WITH PORTLAND CEMENT CONCRETE

Properties	High Impact Flexible Polyester Mortar	Portland Cement Concrete (16, 17)
Compressive strength (psi)	4,900 ± 100	2,000 - 5,000
Modulus of elasticity (psi)	$7.1 \times 10^5 \pm 2.9$	$2.4 - 4.9 \times 10^6$
Tensile strength (psi)	556 ± 69	200 - 500
Average air flow rate[1] (ml/min)	2.28 ± 0.72	--

[1] Two-inch diameter area, pressure differential 1.00 in. water.

TABLE 5

SOLVENT RESISTANCE OF POLYESTER MORTAR

Soaking Time at 70 F (hr)	Compressive Strength (psi)		
	After Soaking in Jet Fuel, Type A-1[a]	After Soaking in Auto Gasoline	After Soaking in Aviation Hydraulic Fluid
0	4,900	4,900	4,900
5	4,690	4,675	4,725
24	3,375	3,625	5,125
96	4,840	5,390	5,350

[a] ASTM Designation D 1655-61T.

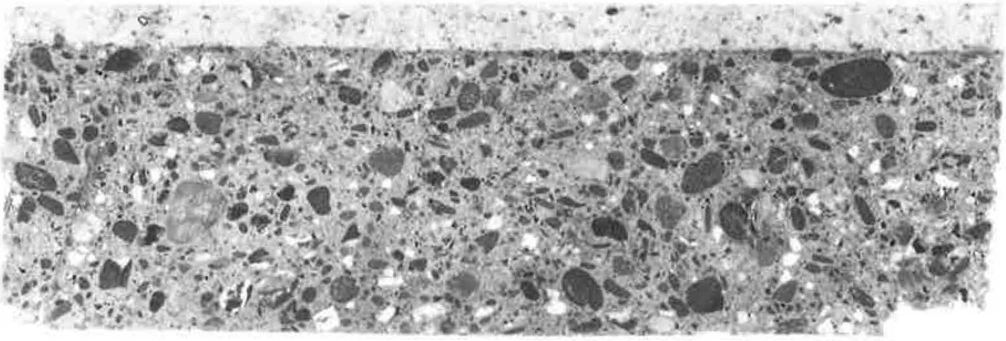

Figure 5. Polyester mortar tightly bonded to PCC section with polyester prime coats.

The effect of temperature variation on the bond strength of the mortar overlay was studied on the sliced section shown in Figure 5. This ¼-in. thick sample was heated to 425 F and immediately placed in a 39.2 F bath. Microscopic inspection showed no signs of bond separation. A similar sample was fractured with a hammer after this test. In no case did the fractured pieces show failure at the overlay-concrete interface.

The polyester mortar when properly prepared can attain high compressive strengths in a relatively short time. Once the accelerator and catalyst have been added to the polyester resin, a chemical reaction begins. Curing time of the resin can be readily adjusted from a few minutes to several hours. The resins used in the field trials were designed to set in about 1 hr at normal room temperature. The increase in mortar strength with curing time is shown in Figure 6. The mortar is shown to reach 80 percent of its ultimate compressive strength within 24 hr at ambient temperature and in 17 hr is stronger than most fully cured PCC (15). This high early strength allows coated PCC surfaces to be opened to traffic shortly after construction.

CONSTRUCTION OF POLYESTER OVERLAYS

PCC surfaces are prepared in a similar way for both seal coats and the mortar system. All deteriorated concrete is first removed to expose sound surfaces. A solvent wash is used to remove asphalt and oil spots. This is necessary for best adhesion and because most asphalts have a retarding effect on polyester cure. The surface is then prepared for the overlay by sandblasting or by etching with a dilute acid solution. The method used is largely dependent on the condition of the PCC. In the case of an acid etch, the deck is flushed with water and must be thoroughly dried before applying polyester resin because water also affects the wetting out, adhesion, and cure of the resin.

The conventional seal coat is constructed by imbedding suitable chips into an appropriate amount of polyester resin. The resin rate and aggregate size and type are selected for the final surface texture desired.

The mortar overlay is bonded to the concrete base with a combination of two polyester prime coats. The first primer

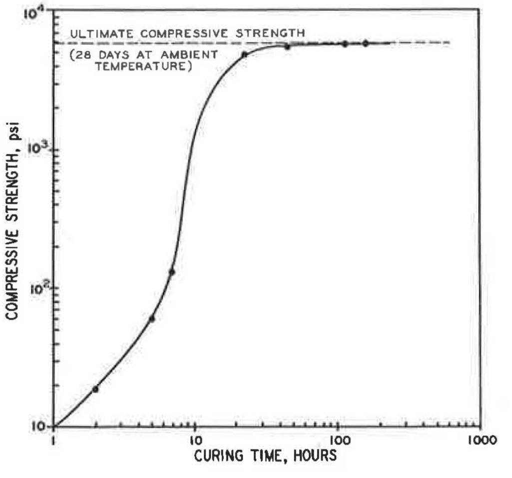

Figure 6. Increase of compressive strength of mortar with curing time.

is lower in viscosity and designed to penetrate and seal off the voids in the PCC surface, thus providing "in-depth" adhesion for the mortar to the concrete. The next prime coat is the same resin as that used in the mortar formula and acts as the major bonding medium between the mortar and concrete. The blended and catalyzed mortar is next roughly troweled or screeded to the desired thickness over the freshly primed surface. Compacting the mortar to maximum density completes the placing process.

To prevent sticking of the polyester mortar to the compacting roller used on the relatively small size field trials, a plastic film was passed between the roller and mortar. After compaction the film is peeled from the mortar leaving a slightly rippled surface texture. These ripples are probably caused by slippage of the film under the roller.

Additional skid resistance is imparted to the overlay by brooming the surface just as the mortar begins to cure (Fig. 7). The final texture of a broomed surface is shown in Figure 8. A still rougher texture is obtained by rolling a liberal coating of angular chips into the uncured mortar. Excess chips are swept from the surface after the mortar has hardened. Such a surface texture is shown in Figure 9.

SELECTION OF MATERIAL AND APPLICATION RATES

The seal coat overlay can be designed to give any desired type of surface. If a fairly rough textured surface is needed, the seal coat is made with $^3/_8$-in. chips. For a fine textured overlay, No. 20 or No. 30 mesh grits are bonded to the PCC deck with ap-

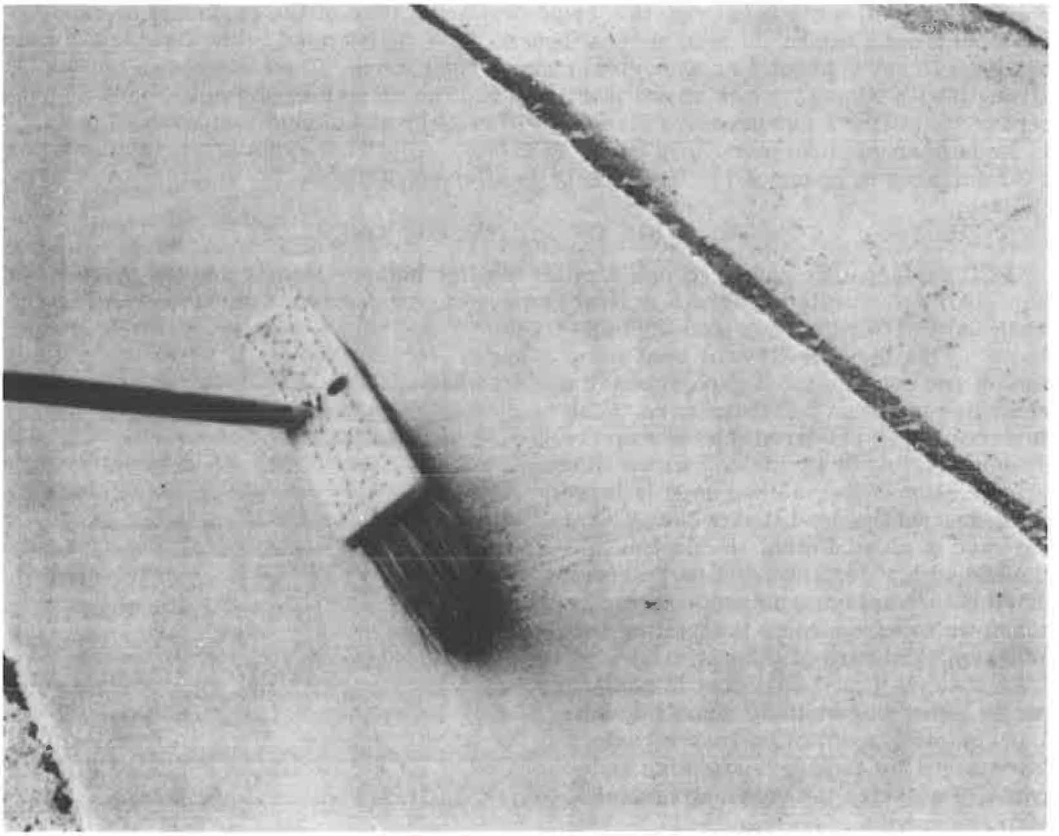

Figure 7. Brooming just before cure to give polyester mortar toughened skid-resistant surface.

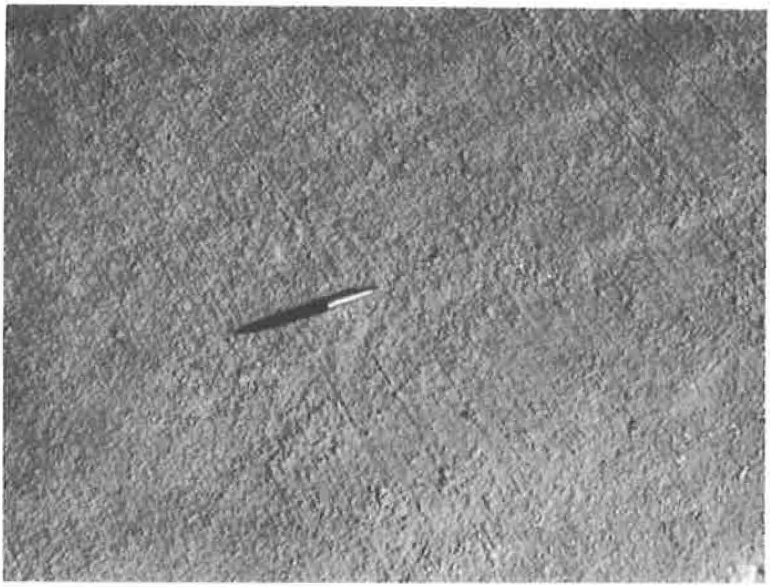

Figure 8. Final surface texture of broomed mortar section.

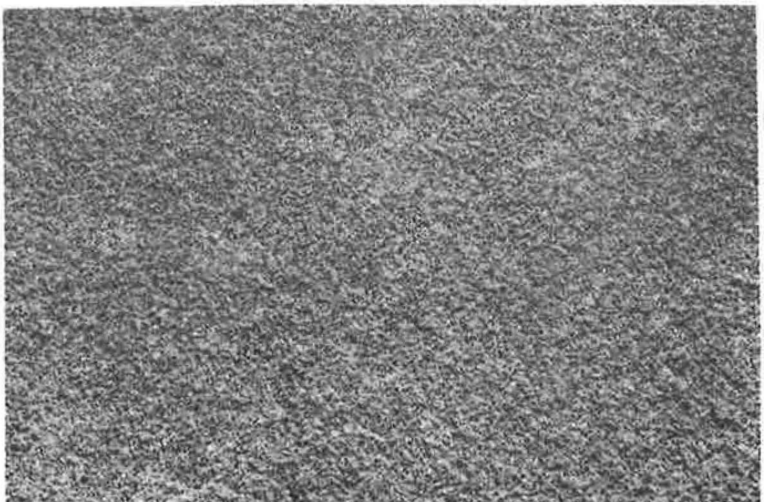

Figure 9. Stone chips in mortar overlay to increase skid resistance.

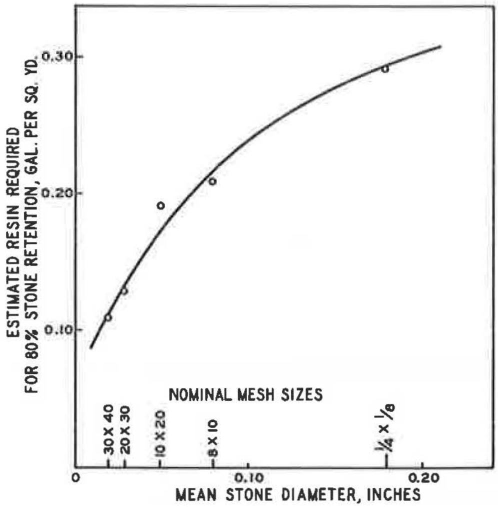

Figure 10. Relation between resin requirements and stone size for 80 percent stone retention after 3 yr of service on PCC surface.

proximately 0.13 gal of resin per sq yd (1.14 lb per sq yd). Typical application rates for maximum stone retention, based on the field studies, are shown in Figure 10. These rates are included only as guides because the porosity of the PCC surface will often dictate resin requirements.

The aggregate used in the mortar overlay system is controlled by the desired thickness of the coating. For ¼-in. overlays, a No. 8 mesh sand is chosen as the maximum size aggregate. The recommended maximum size aggregate is one-half the thickness of the desired overlay.

The application rates for the prime coats used with the mortar overlay range from 0.06 to 0.08 gal per sq yd (0.53 to 0.70 lb per sq yd). This is again dependent on the porosity of the PCC surface and the viscosity of the priming resin.

CONSTRUCTION EQUIPMENT

Readily available equipment can be used for the major construction of polyester seal coats. For limited areas, the resin can be sprayed on by hand or with a self-propelled spray rig (11). Where larger sections are to be coated, a dual spray bar unit equipped with multiple spray heads may be used. Stone chips may be spread by tail gating from a truck or with commercially available chip spreaders.

The prime coats used with the mortar overlay system can be sprayed on with the methods described for seal coats. A 40-in. long, 12-in. diameter hand steel roller weighing 35 lb per linear in. was used to compact the mortar placed in the field trials. For full-scale construction, any suitable self-propelled unit may be used.

PERFORMANCE OF POLYESTER OVERLAYS

As discussed earlier in this paper, the polyester seal coat installations have performed well under heavily loaded truck traffic for over 3 yr. The high impact flexible polyester mortar overlay trials in the Richmond Refinery are also in excellent condition after 9 months of traffic.

In-place tensile tests (5) were run to evaluate the bond strength of polyester mortar overlays. All the tensile tests on high impact rigid polyester mortars resulted in adhesive bond failures at the concrete-overlay interface. When a high impact flexible resin was used in the mortar, the bond strength increased but still showed up as partial cohesive failure in the concrete and partial adhesive failure at the bond line. In both sections only one prime coat was used. For a high impact flexible polyester mortar bonded with multiple prime coats, tensile tests gave even higher bond strengths with complete cohesive failure in the concrete. Figure 11 shows this type of failure. Test results are shown in Figure 12. The tensile values reported represent an average of at least three tests. The comparison of results just described is for an acid-etched PCC surface.

Similar tests were run on sandblasted areas on the same PCC deck (Fig. 12). This particular concrete surface was in good condition, but conceivably poorer PCC surfaces would show marked differences between acid etching and sandblasting as surface preparatory methods.

RELATED USES OF POLYESTER SYSTEMS

Seal coat applications of polyester and stone will be beneficial wherever corrosion

Figure 11. Typical cohesive bond failure in concrete.

control or maintenance of PCC surfaces is a problem. Thus, in addition to highway use, seal coats of this type may find use as coatings on commercial floors, wharf decks, and storage reservoirs.

The mortar overlay also has numerous potential uses as a protective coating. Its benefits as an impermeable solvent-resistant overlay for bridge decks have been demonstrated. In addition, the mortar may find use on PCC surfaces subjected to severe deterioration by wave action or turbulent flow. The mortar could be applied to buckets and faces of spillways, the upper face of concrete dams, bridge piers, or the interior of concrete canals.

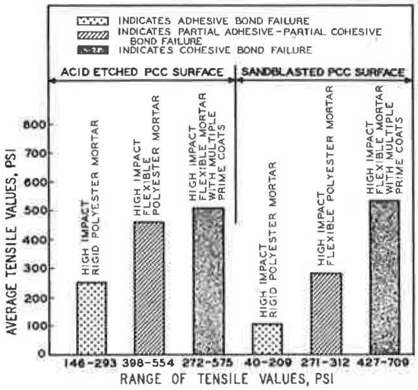

Figure 12. Tensile test results showing effect of resin-type and surface preparatory methods.

CONCLUSIONS

Polyester seal coats have demonstrated their use as durable protective coatings under heavy vehicles. The texture of the seal coat surface can be controlled by proper selection of aggregate particles.

Initial field trials have shown polyester mortar overlays to be a promising material for protecting PCC surfaces from environmental deterioration. When properly designed and applied, the mortar provides the following qualities:

1. An impermeable surface which prevents infiltration of water, solvents, and deicing salts into the concrete base.
2. A thin coating which is essential in resurfacing bridges where additional dead weight must be kept to a minimum.
3. A leveling material which allows for the realignment of irregular or deteriorated PCC surfaces without significantly raising the grade.
4. Fast curing which eliminates "out-of-service" traffic delays. The mortar has been shown to develop strengths greater than most concretes in less than 20 hr.

5. A light-colored overlay that can be pigmented for any desired effect.
6. Antiskid properties for highway safety.

Although the initial cost of polyester resins is high compared to those of cement and asphalt, the savings in maintenance and repair of PCC surfaces may far offset the difference. Moreover, thermosetting resins, like polyester, provide strength, chemical-resistant, and wear-resistant properties superior to existing bridge coatings.

ACKNOWLEDGMENTS

The author acknowledges the support of the Oronite Division of the California Chemical Company who sponsored this work. The author also appreciates the technical assistance provided by the Coatings and Plastics Chemicals Technical Service Group of California Research Corporation.

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Discussion

H. B. BRITTON, New York State Department of Public Works—The author is to be congratulated for the excellent manner in which he has developed his subject. The details and graphic description contained in this paper could only result from an intimate knowledge based on experience.

It is interesting to note the influx of new and sophisticated materials for the purpose of protecting, restoring or developing concrete in order that its serviceable life might be increased.

The paper presented outlines two methods of employing polyester resin as an overlay protective system for portland cement concrete. These have been designated as a seal coat and a mortar overlay.

It is recommended that the seal coat be used only on new concrete or on concrete that is relatively free of surface deterioration. It is not definitely stated, but it is inferred, that the seal coat is established by a single application of the polyester resin on which chips of a gradation commensurate with the finish desired are spread. This procedure might well raise the question as to whether the final product had produced a truly impermeable material inasmuch as of necessity the wetting out of the chips could cause a reduction in the final mil thickness of the resin. Because it has been demonstrated that a prime coat of polyester resin enhances the properties of the mortar overlay, it would appear reasonable to expect the same results if this procedure was used when applying the seal coat.

In the development of the mortar overlay, the rigid polyester resin was replaced with a flexible polyester resin. The author presents a choice as to the reasons for the failure of the mortar overlay made with the rigid polyester resin. It might have been helpful if some definite conclusion could have been drawn relative to the failure.

The gradation and percentages of the aggregates for use in the mortar overlay, though possible to attain in the laboratory, might be impossible to realize under field conditions except at elevated or prohibitive cost. The proportions given in Table 3 are predicated on a perfect nesting of each successive graded aggregate into the interstices of its predecessor. This ideal of void satisfaction is difficult to accomplish because particle shape is as much a factor as particle size.

It would also appear from the proportions in Table 3 that the ratio of filler to the resin is too high; in fact, it could possibly result in robbing the complete system of its resin requirement.

From the indicated air flow rate in Table 4 it is obvious that the mortar overlay is porous in nature. This may be due in part to the observations previously noted.

This indicated porosity would preclude the use of the mortar overlay in a great many of the States where the incidence of freeze-thaw cycles is high.

Considerable detail was given to the description of methods of placement and equipment; however, no mention is made of the equipment required to produce the mortar. It is assumed that conventional concrete mixing equipment can be used.

The two systems outlined in this paper show considerable promise and should be given the opportunity to demonstrate their ability to perform under various construction conditions. One such condition should be on new construction and the other as a maintenance operation.

When utilized in new construction, the designer should be cognizant of this fact and note on his plans the desired finish texture of the concrete best suited to the polyester application.

Under maintenance operations, surface conditions should be attained as recommended in the author's paper. Under maintenance conditions deteriorated concrete must necessarily be removed, and the paper has demonstrated the practical advantages of choosing the mortar overlay as a leveling course. It would then appear that the best results would be obtained by limiting the use of the mortar overlay for this purpose and follow with a complete application of the seal coat. This would then provide a system that would make possible a 100 percent runoff of water or solution from the top surface without the possibility of migration into or through the system.

Because compaction of the mortar is recommended by the author and a relatively smooth riding surface area is desirable, would it be possible to provide polyethylene sleeves for the rollers to overcome the slippage encountered in the use of the plastic sheet?

It would be most helpful if a time interval would be indicated for the successive applications of the resin. On this time differential will depend the type of bond obtained between succeeding coats of resin. If the time interval between succeeding coats is in the area of one to four hours, there is a good possibility that the bond could be effected by copolymerization. If the time interval between succeeding coats is in excess of a day it could be that the bond produced will be mechanical.

The indication that both the mortar and its methods of application are claimed in patent application now pending before the U. S. Patent Office is the only disturbing statement contained in the paper. This might develop a reluctance on the part of many end users to investigate the potential of this apparently desirable material.

In presenting this formal discussion the writer is playing the role of the second guesser and anything contained herein should not be interpreted as intending to detract from this very fine work done by the author.

L. E. SANTUCCI, Closure—The author wishes to thank Mr. Britton for his very appropriate comments on the paper. He has brought out several points about the paper which require further clarification or re-emphasis at this time.

First, the author would like to emphasize the fact that both the polyester seal coat and the mortar overlay discussed in the paper are completely impermeable to water. Several field tests were conducted in which a 4-in. diameter glass tube was tightly bonded to the overlay surface. Water was then added to the tube to an initial height of 6 in. After three days of standing, no measurable drop in the water height was noted. Furthermore, the permeability results reported in this paper are air permeability values, measured under a pressure differential of 1 in. of water. The fact that an absolutely zero value vs the reported value of about 2.3 ml per min was not obtained is probably due to the limitations of the air permeability device to measure completely impermeable samples.

In measuring air flow rate, the sample, whether it be an asphalt concrete core or a $^1/_4$-in. thick plug of the mortar overlay, is placed in a "core adapter" described in "A Method for Measuring the Air Permeability of Asphalt Concrete Pavements," ASTM Special Tech. Publ. 294 (1960). The adapter consists of a rubber membrane which is collapsed around the circumference of the sample. Often, with highly impermeable samples, the air flow rate obtained may be the result of a leak around the edge of the sample rather than passage of air through the specimen.

In answer to the question on the choice of reasons for failure of the high impact rigid polyester sections, the author was actually unable to isolate either of these reasons as being the major cause of failure on this test section. However, it is felt that there is a better than 50 percent chance that failure was caused by excessive stresses set up in the rigid polyester resin during the curing cycle. Nevertheless, dropping the modulus of elasticity slightly below that of concrete will reduce the chance of failure by cracking.

The aggregate gradation of the mortar given in Table 3 is actually the grading used in all the field installations. No particular difficulty was encountered in placing the material. However, it should be recognized that the mortar is a relatively dry mix and does require more than simple troweling.

The proportion of filler to resin used in the mortar may be expressed in another way. For example, the filler to binder (F/B) ratio, in this case the polyester resin, is 0.59 on a volume basis. This value falls within the recommendations of Nijboer (14) in his asphalt concrete mix design method. Nijboer suggests a desirable range of F/B ratio of 0.3-0.6. Because the polyester resin acts essentially as an asphalt in bonding aggregate particles together, there is no reason for not accepting these recommended proportions.

The author wishes to apologize for not including the type of equipment used in preparing the mortar. Actually, Mr. Britton's assumption is correct in that typical paddlewheel type of mixer was used to blend the graded aggregate and catalyzed polyester resin before applying it to the test surface.

TABLE 6

ESTIMATED COST IN PLACE OF POLYESTER SEAL COAT AND MORTAR OVERLAY

Types of Costs	Polyester Seal Coat		Polyester Mortar Overlay	
	\$/lb	\$/Sq Yd	\$/lb	\$/Sq Yd
Resin:				
As bond coat, 2.0 lb/sq yd	0.35[a]	0.70		
As primer, 1.1 lb/sq yd			0.35[a]	0.39
In mortar, at 2.6 lb/sq yd			0.35[a]	0.91
Sand:				
(10 by 20 mesh)				
At 18 lb/sq yd	0.01	0.18		
Special Calresearch sand Blend, 23 lb/sq yd			0.01	0.23
Miscellaneous[b]:		1.57		1.72
Total cost		2.45		3.25

[a]Based on average current polyester resin cost, Chem. and Eng. News, Quarterly Report on Resin Prices (May 7, 1962).
[b]Including surface preparation, equipment and labor, profit, and contingencies.

As to the use of polyethylene sleeves on the roller, the author conducted several small-scale field tests in which various release compounds such as silicone and plastic coated rollers were tried. None of the release materials prevented sticking of the mortar to the steel wheel. Teflon- and polyethylene-coated rollers showed no sticking of mortar to the roller for one or two passes. After this, however, small pieces were picked up which quickly snowballed into a major pickup of material. This problem was not solved by adding a scraper bar to the roller. The use of a polyethylene film as a physical barrier between the compacting roller and the fresh mortar has proved to be the most successful approach. Because the mortar surface will most likely be roughed by brooming or adding stone chips, the author has not been concerned with the rippled surface pattern caused by slippage of the polyethylene sheet under the roller.

The question on the time intervals for successive applications of the resin is a very good one. The diluted polyester prime coat is first applied to the cleaned PCC surface. Shortly thereafter, approximately 5 to 10 min, the second polyester prime coat is placed on the deck. The blended and catalyzed mortar is immediately spread over the uncured prime coats. As a result, there is no time delay in waiting for one layer of application to set up before the next phase of the overlay system is added.

The author feels certain that Mr. Britton understands the author's position regarding patent applications. As a producer of new products or novel ways for using existing products, it is often necessary to protect the proprietory position of such an invention by patent coverage. It is only fair to inform end users that patent applications on both the mortar and its method of application are pending before the U. S. Patent Office.

As a result of several questions on the estimated cost of the two polyester overlay systems described in the paper, the author refers to Table 6. The costs in the table are given as estimates only, and it should be realized that variations in the quoted figures can result from (a) the condition of the PCC surface to be coated and, hence, the amount of surface preparation required, (b) the porosity of the PCC surface, (c) local labor costs, (d) the availability of locally suitable aggregate, and (e) the type and current prices of the polyester resin selected for the mix.

Environmental Influence on Corrosion of Reinforcing in Concrete Bridge Substructures

J. L. BEATON and R. F. STRATFULL, respectively, Supervising Highway Engineer and Corrosion Engineer, Materials and Research Department, California Division of Highways

Prompted by evidence of corrosion which occurred in the reinforcing steel used in 20 highway bridges in an arid desert within 10 years after construction, a survey was made of the condition of 239 bridges located throughout the State of California. The survey indicated that corrosion of the embedded steel was evident in varying amounts in approximately 28 percent of the bridges. These structures ranged in age up to 50 years.

It was found that bridge deterioration was related to chloride content of the soils or waters in the environment. An equation was derived for the percentage of structures that had deterioration for any concentration of chlorides in the immediate area.

Laboratory tests continuing over a period of 2½ years indicated a relationship between concrete's cement and water contents and its ability to gain and lose water vapor in controlled environments. Using the variables that appear to influence the movement of water vapor through concrete, an equation was developed which gives the probable time to cracking of a reinforced concrete substructure. The equation is based on field observations and laboratory tests, and takes into account the effect of the variables of cement and mixing water content, the thickness of cover over the steel, and the chloride content of the environment.

•HUNDREDS of reinforced concrete bridges have been constructed over the past 50 years in the State of California. The majority of these structures have required a minimum of repair attributable to the corrosion of reinforcing steel. However, there are a number of bridges showing evidence of cracking due to steel corrosion.

In California the most serious example of costly maintenance has been the deterioration repair of the San Mateo-Hayward Bridge (1). Although the deterioration of this structure has received considerable attention from the California Division of Highways, the distress of numerous reinforced concrete structures exposed to marine environments is not unusual (2). In a 1917 report (2) of the inspection of 146 structures on the east and west coasts of the United States, it was observed that "concrete can be used successfully in sea water, but the price of success is eternal care." This statement was based on existing technology as applied to a concrete coverage of 1 or 2 in. over the reinforcing steel.

Paper sponsored by Committee on Metals in Highway Structures.

In addition to the 1917 inspection of American-made structures, an inspection by the Committee of the Institution of Civil Engineers on structures in British waters in 1920 also revealed deterioration of numerous reinforced concrete structures (3). More recent reports (4, 5, 6) have shown that distress of reinforced concrete structures in marine environments continues. On the basis of these reports and the experience in California, it appears that due to corrosion of steel the cost of maintaining these structures may be expected to be extraordinary.

Previously, the investigation of the corrosion of reinforcing steel by the Materials and Research Department of the California Division of Highways has been primarily directed toward determining the effect of marine environments on the durability of reinforced concrete. However, in 1959, an investigation (7) was made of the distress of a 9-year-old highway bridge constructed in the arid Colorado Desert of California. It was found that this structure had the same evidence of distress as the San Mateo-Hayward Bridge (1) across the San Francisco Bay. On this same State highway, 66 reinforced concrete bridges were constructed during the years 1950-51, and 20 of the 66 bridges (about 30 percent) were found to have corroding reinforcing steel. Data collected during this investigation showed that the high concentration of salts in the soil (flow is usually not present at these bridge sites except once or twice a year) was responsible for the observed distress of the structures. Bridges in soils of high salt concentrations (up to 41,000 ppm of chlorides) showed distress, whereas those in low salt concentration (less than 200 ppm) were in a satisfactory condition.

Because of these findings on corrosion of reinforcing steel in an environment other than marine, a statewide investigation was made to determine the scope of this problem. This paper presents the results of this investigation.

CONDITION OF BRIDGES

The types of environments in which bridges in California were inspected for possible corrosion of reinforcing steel were (a) coastal, (b) valley, (c) desert, and (d) mountain. The 239 bridges inspected varied in age up to 50 years. The survey of the structures was visual and was concerned with determining the incidence of concrete cracking parallel to the reinforcing steel or rust stains on the surface of the concrete. These criteria were considered indicative of the corrosion of reinforcing steel. Typical examples of these observed conditions are shown in Figures 1 and 2. Figure 1 shows the cracking of reinforced concrete piles in a 10-year-old bridge exposed to tidal water near San Diego, Calif.; Figure 2 shows an area of advanced concrete spalling and the exposed reinforcing steel on the San Francisco-Oakland Bay Bridge.

Of the 239 bridges inspected, 66 showed evidence of corrosion of the reinforcing steel in some member of the structure. For the purpose of this investigation, only the reinforced piling, piers, or walls were considered to be directly affected by the variables found there. Distress observed in the rail, beams, etc., of the bridge could conceivably be influenced by air-borne salts in the atmosphere or by other atmospheric variables. Distress in these members was therefore considered outside the scope of this investigation. Based on this criterion, 37 structures showed visual evidence of corrosion of reinforcing steel in a pile or wall that was in direct contact with the natural soils or waters. The 29 structures that had visual evidence of deterioration in members other than the substructure were not used in the analysis that follows. Also, 5 structures which had less than 4 years of service were not used in the analysis.

In addition to the visual inspection of the condition of the structures, the following environmental data were collected for a possible correlation to the condition of the structures: (a) pH or hydrogen-ion concentration, (b) sulfates as SO_4, (c) chlorides as Cl, contained in the soils and the natural drainage waters, and (d) the electrical resistance of the soils and drainage waters.

CONCRETE DISTRESS

Chlorides

In the analysis of concrete distress due to the corrosion of reinforcing steel, it was not possible to determine when the crack in the concrete occurred. The only fact known

Figure 1. Vertical concrete cracking of 10-yr-old reinforced concrete bridge piles in tidal water near San Diego, Calif., 1962.

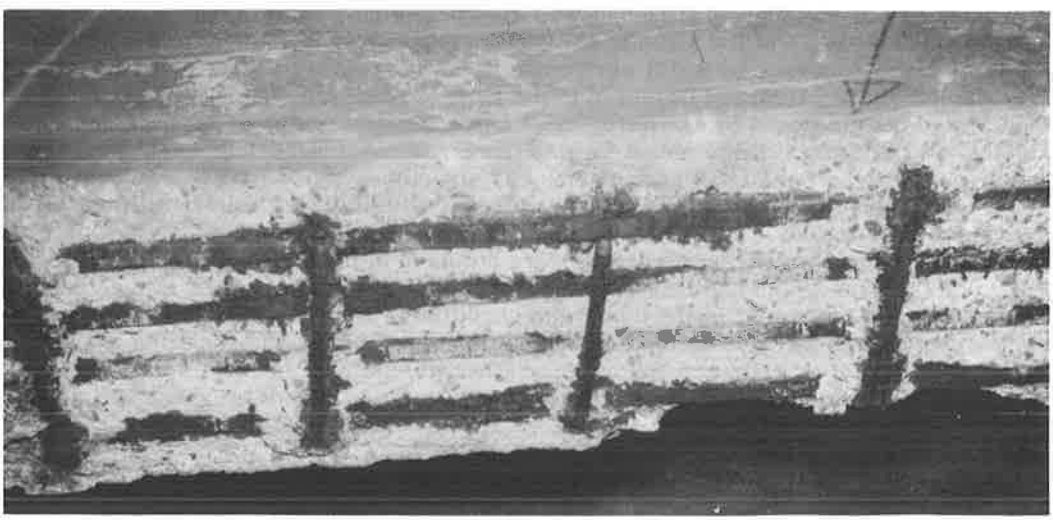


Figure 2. Spalling of concrete in beam caused by corrosion of reinforcing steel, San Francisco-Oakland Bay Bridge, 1962.

about the distress was that at the time of the inspection of a structure a crack was or was not present. Therefore, the data were analyzed by mathematically grouping the bridges into age vs deterioration and also vs the environmental factors.

Figure 3 is an example of the method used for plotting the "raw" data. The structures were first segregated into average age groups of 10, 20, 30, and 40 years of service. Then a plot was made of the condition of the structures at the particular

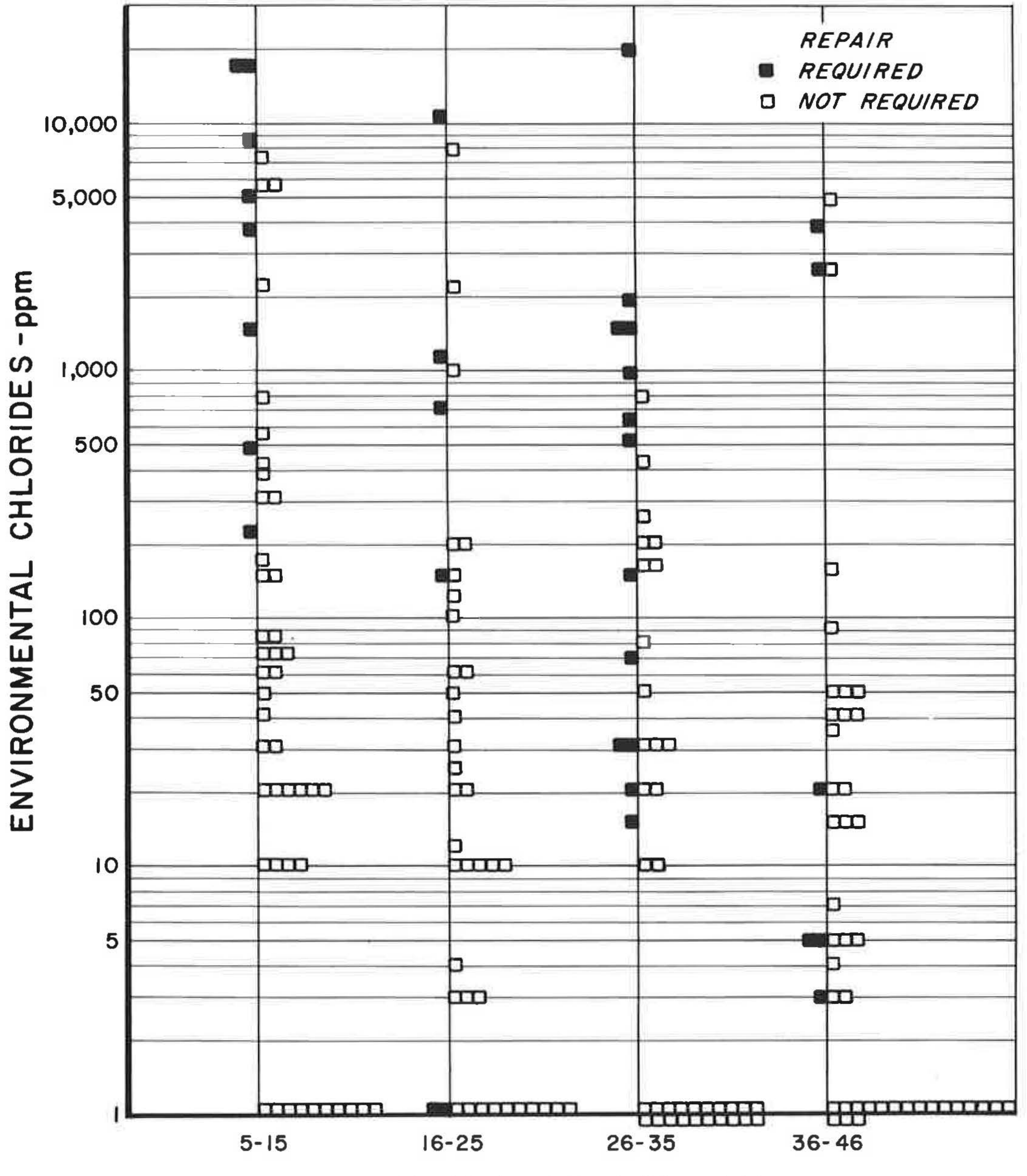


Figure 3. Influence of chlorides on deterioration of reinforced concrete bridge piles or walls.

chloride concentration of the environment. The figure shows that there is a relationship between the chloride concentration of the environment and the distribution of the condition of structures.

There were 29 structures that had visual evidence of deterioration in other than substructure members, 5 nondeteriorated structures with less than 4 years of service, 15 bridge locations in which a chemical analysis of the soil was not obtained, and 4 structures in which deterioration was observed after the initial investigation; these preceding data were not included as a part of the original data reduction. However, with judgment, some of the preceding data were included in Figure 10, discussed later.

The data (Fig. 3) were then put into distribution curves of the percentage of structures deteriorated at each of the four time increments. They were then mathematically analyzed to determine the yearly rate at which the structures would deteriorate in each chloride concentration. The result of the analysis is shown in Figure 4.

The correlation equation is

$$R_y = 0.165\ K^{0.42} \tag{1}$$

in which

R_y = percent of total structures showing distress per year; and

K = chloride concentration in parts per million found in either the soil or water that is in physical contact with the substructure.

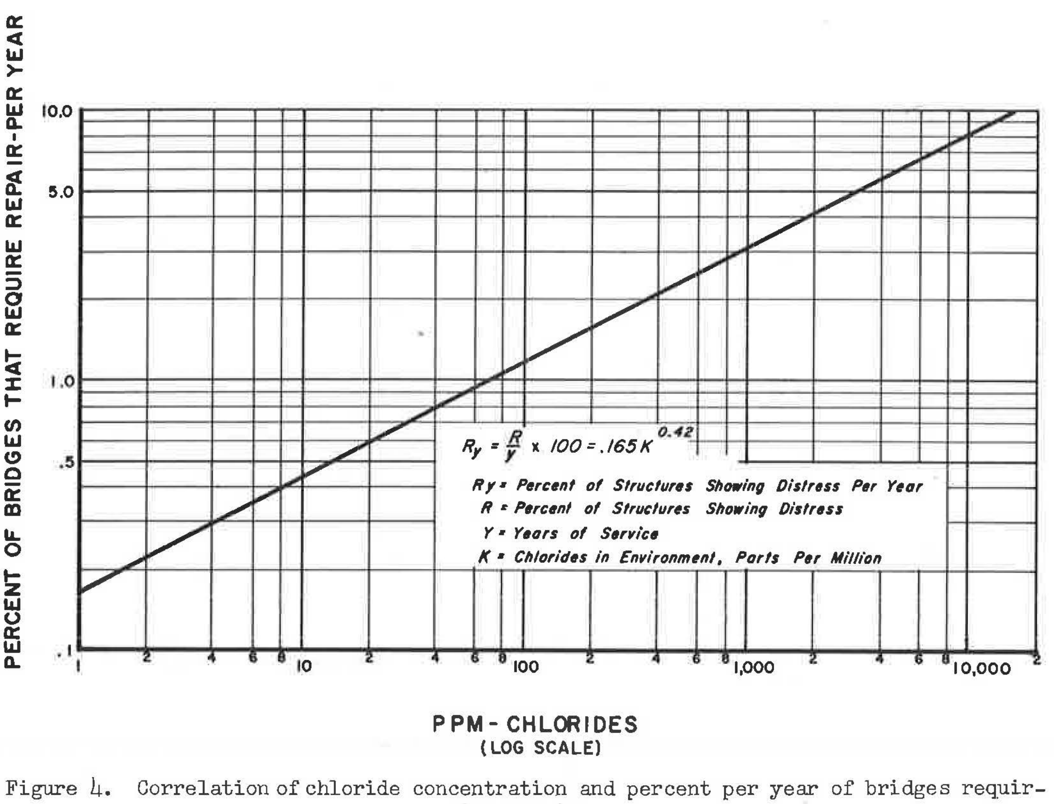


Figure 4. Correlation of chloride concentration and percent per year of bridges requiring repair.

After the equation was derived, it was tested for its ability to duplicate the original distribution curve of the condition of the bridges.

The method of least squares showed that there was a correlation coefficient of 0.957 and the standard error of estimate was 7.08 percent. This indicated that the results computed by use of the derived equation were within 7.08 percent of the actual percentages of structures found to be in distress when inspected. The level of significance of the correlation is greater than 0.001.

The data represented by Eq. 1 are typical of a distribution curve of the observed condition of a number of bridges. Analyzing the performance of a single structure would require a thorough investigation of all variables that might cause a deviation from the mean. Without doubt, there are variations in the relative protective value of concrete made throughout the years as a result of differences in environment, workmanship, curing, etc. For example, these data are for visible sections of a bridge substructure. The incidence of deterioration below ground or under water was not considered.

The concrete normally used for the construction of bridge piles consisted of 6 sacks of cement per cubic yard of concrete, and the design provided 2 in. of concrete cover over the reinforcing steel.

Sulfates

As previously stated, the sulfate (SO_4) contents of the soil and water, if any, at the bridge sites were determined and mathematically computed in the same manner as the chloride contents.

The analysis indicated that there was correlation between the sulfate content in the environment and the percentage of structures showing distress. However, it was generally observed that, when a high sulfate content was found in the environment, there was generally a high chloride content as well.

This observation suggested that there could be an interdependence between the chloride and sulfate concentration and the deterioration rate of the structures. To determine the possible interdependence of the chlorides and sulfate concentration and bridge distress in the environment, the data obtained for sulfates were analyzed for a correlation to bridge conditions in the same manner as for chlorides. Then the quantity of sulfates was plotted against the rate of bridge deterioration at each age group. At each of these same points of rate of bridge deterioration and age groups, the concentration of chlorides and sulfates was tabulated. The results of this comparison (Fig. 5) indicate that the more imminent cause of reinforcing steel corrosion is the chlorides present in the environment.

CONCRETE VARIABLES

Mixes

In addition to the study of the correlation between the chloride concentration in the environment and the condition of the bridges, studies were made of the vapor transmission characteristics of various types of concrete.

The corrosion of steel in concrete appears to be caused by the deposition of salts adjacent to the steel as a result of moisture movement. Therefore, the influence of the variables in mixtures was compared to the relative ability of the concrete to gain and lose moisture. The concrete test blocks were alternately exposed for periods of about four months (a) in a fog room at 73.4 F and 100 percent relative humidity, and (b) to drying at 73.4 F and 50 percent relative humidity. It was observed that the 3- by 3- by 11.25-in. concrete blocks would approach moisture equilibrium after three to four months of exposure.

In all cases, three concrete specimens were made with the following variables: (a) aggregate having 1.5, 4, and 10 percent absorption by weight, (b) equivalent of 4, 6, and 8 sacks of Type II cement per cubic yard, and (c) slump of 2, 4, and 6 in. The maximum aggregate size was 100 percent passing the ¾-in. screen. In addition, neat cement bars of 1 by 1 by 11.25 in. were cast with the equivalent of about 31, 32, 34, and 35 sacks of cement per cubic yard and the resulting neat cement flows of these mixes were 10, 37, 56, and 56, respectively.

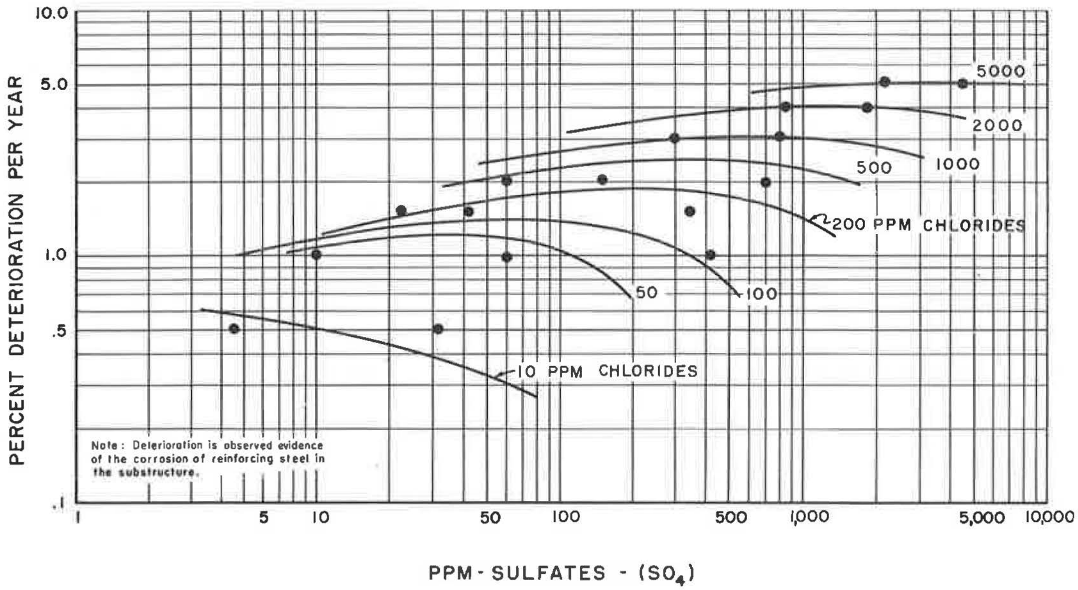


Figure 5. Influence of sulfate and chloride concentration on bridge deterioration.

All the concrete blocks and neat cement bars were cured for seven days in the fog room and then placed in the dry room for the beginning of the first dry cycle. The following data for the concrete blocks were obtained after approximately 2½ years of alternately exposing the concrete to the fog and drying room atmospheres. The study of the concrete blocks is continuing.

Water Voids

Various methods have been used to express the quantity of water gained and lost by concrete (8); however, this investigation was primarily concerned with comparing the gain and loss of moisture by various concrete mixtures under conditions comparable to a natural environment. Also, the measurement of the gain and loss of moisture was to be on a continuing basis in order to study the variable of continued concrete hydration; therefore, none of the concrete specimens has as yet been subjected to oven drying.

For this reason the term "water voids" has been used to describe the volume of water absorbed and evaporated by concrete that is subjected at 73.4 F to the alternate conditions of 50 and 100 percent relative humidity. None of the concrete specimens was subjected to other than the described environmental conditions except for the rare occasions when it was necessary to adjust the environmental rooms mechanically.

The weight of moisture gain and loss in the concrete specimens was measured to the nearest one gram during each cycle of exposure at time intervals of 1, 2, 4, 8, 16, etc., days. This was continued until the data indicated it would take more than one month of exposure for the concrete to have a weight gain or loss of one gram. Generally, this condition of moisture stability was obtained within approximately 128 days, or about four months of exposure.

After approximately one year of the alternate exposure of the concrete to fog wetting and drying, it was observed that the weight of water gained and lost for each triplicate set of concrete blocks was approaching a reproducible value. Therefore, the measurements of moisture movement considered significant in this study are for a period of exposure of the specimens that begins after the concrete has aged approximately one year. Concrete older than one year was termed "mature."

It has been previously mentioned that there were three types of aggregate used in the test with absorptions of 1.5, 4, and 10 percent by weight. The use of this range of aggregates resulted in concrete that varied in weight from about 150 to 110 pcf. Therefore, the measured weight of water gained and lost by the concrete specimens was compared on a volume basis to eliminate misleading data on variations that could occur as a result of extremes in concrete densities.

Figure 6 shows the relationship between the total water used in making the concrete and the volume of water that would be absorbed and evaporated during the environmental exposures of the specimens.

The derived water void equation shows that the relative volume of water gained and lost by the concrete in this test was a function of the total water used in the concrete and the quantity of cement. It was found that either an increase in the added mixing water or an excess contained in the aggregate increased the volume of water that transpired through the concrete for a given cement content. Conversely, for a given total mixing water content, an increase in cement content reduced the quantity of water that was absorbed or evaporated by the concrete under these test conditions.

The equation that shows the measured gain and loss of moisture in mature concrete is

$$V_w = \frac{W}{V} \times 100 = \frac{0.85 W_m^{1.17}}{C^{0.717}} \tag{2}$$

in which

V_w = water voids in percent of concrete volume as measured by loss in drying at 50 percent relative humidity and gain by exposure in fog room;

W = volume of water gained and lost;

V = volume of concrete = AS (area × depth);

W_m = total water contained in concrete mix as percent of concrete volume; and

C = sacks of cement per cubic yard of concrete.

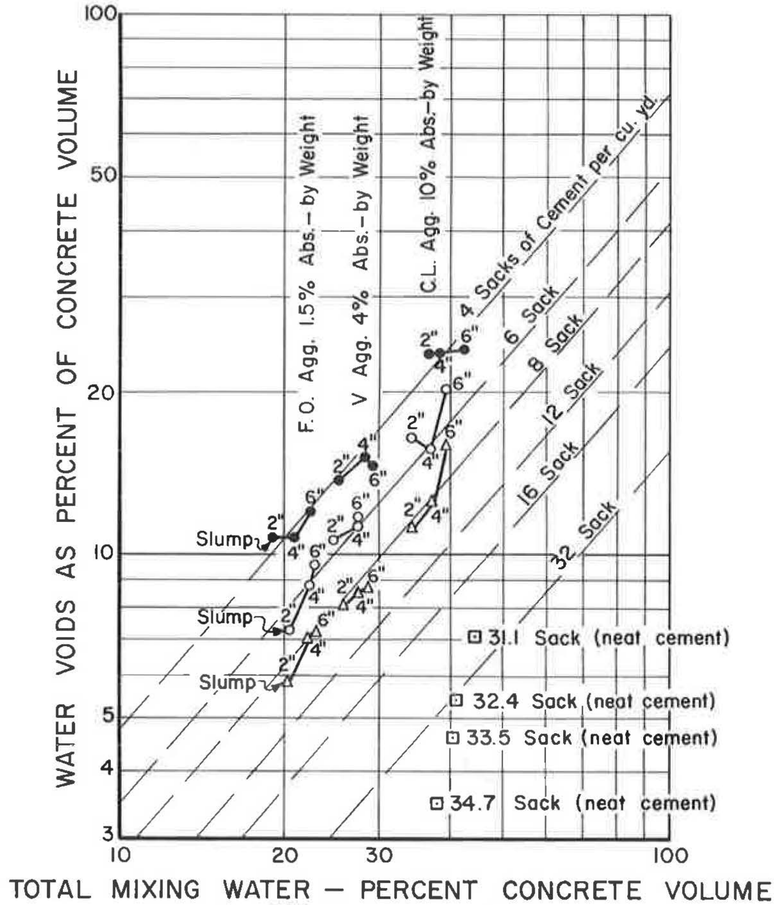
During the first year of measuring the water voids or the volume of moisture that transpired through the concrete test specimens, it was observed (Figs. 7 and 8) that the quantity of moisture movement would not be reasonably duplicated by each subsequent cycle of exposure. Further, under the test conditions it was approximately 200 days after the concrete specimens were cast before a maximum total movement of water would occur. This maximum movement appeared to occur when the blocks were placed in their first exposure to the fog room. The concrete made with the aggregate having 10 percent absorption was the exception.

The variations in the volume of moisture gained and lost during the first year of exposure appeared somewhat disconcerting when the data was analyzed; therefore, these variations were given additional mathematical study.

Figure 7 shows that the cement content and time are the variables responsible for the early differences in the volume of moisture absorption and loss in the concrete specimens. The data show that, with an increasing cement content, the volume of water movement decreases to a minimum value in proportion to the cement content. For instance, the concrete blocks that were made with the equivalent of 8 sacks of cement per cubic yard reduced about 35 percent from their maximum volume of moisture movement, whereas the 4-sack mix reduced about 18 percent.

The aggregates did not appear to exert a major influence on the change in the volume of moisture movement. This fact appears to be shown by Figure 8.

These data indicate that the absorptive and the evaporation characteristics of concrete are highly variable at early ages, and also vary with the cement content and the total water content of the mix.



$$V_w = \frac{0.85\,(W_m)^{1.17}}{C^{0.717}}$$

Vw = Water voids as percent of concrete volume
Wm = Mixing water in percent of concrete volume
C = Sacks of cement per cubic yard

Figure 6. Water voids (computed from volume of moisture gained and lost during prolonged exposure to 73.4 F at 50 and 100 percent relative humidity, respectively) vs mixing water (including that in aggregate) in mature concrete (aged over 1 yr).

Rate of Moisture Movement

During the measurements of the gain and loss of moisture of the concrete blocks, the periodic quantity of water movement was found not to be constant but to decrease with the time of exposure. This observation was not considered unusual inasmuch as there is a limited quantity of moisture that would be available for evaporation. It was also obvious that when moisture had evaporated from the surface of a block, it would be readily apparent by a measurement of weight loss. However, the moisture that migrated from the center of the block toward the surface would not be detected by a weight difference until it evaporated into the atmosphere. Therefore, the rate of moisture change should vary as the distance that the water would be required to travel.

On an empirical basis, calculations were made on the rate and depth of moisture loss with the following considerations: The quantity of moisture that a concrete block contained was known by continuous weight measurements. It was assumed that the moisture was evenly distributed throughout the concrete mass at the end of each exposure period, and in all cases, could be computed for any point within the concrete

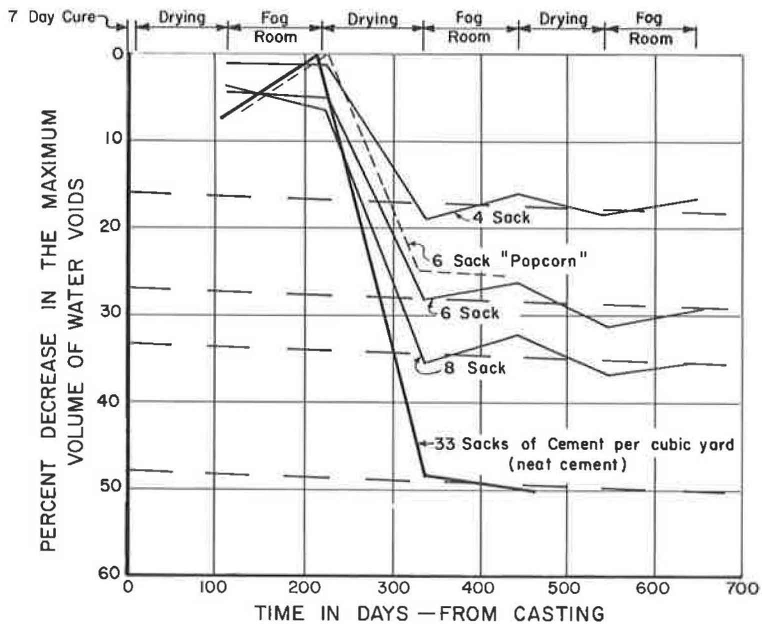


Figure 7. Influence of cement content on volume change of water voids-aggregate constant. Water voids computed from measured volume of moisture gained and lost during indicated exposure; concrete not oven dried; drying and fog room at 73.4 F, and 50 and 100 percent relative humidity, respectively.

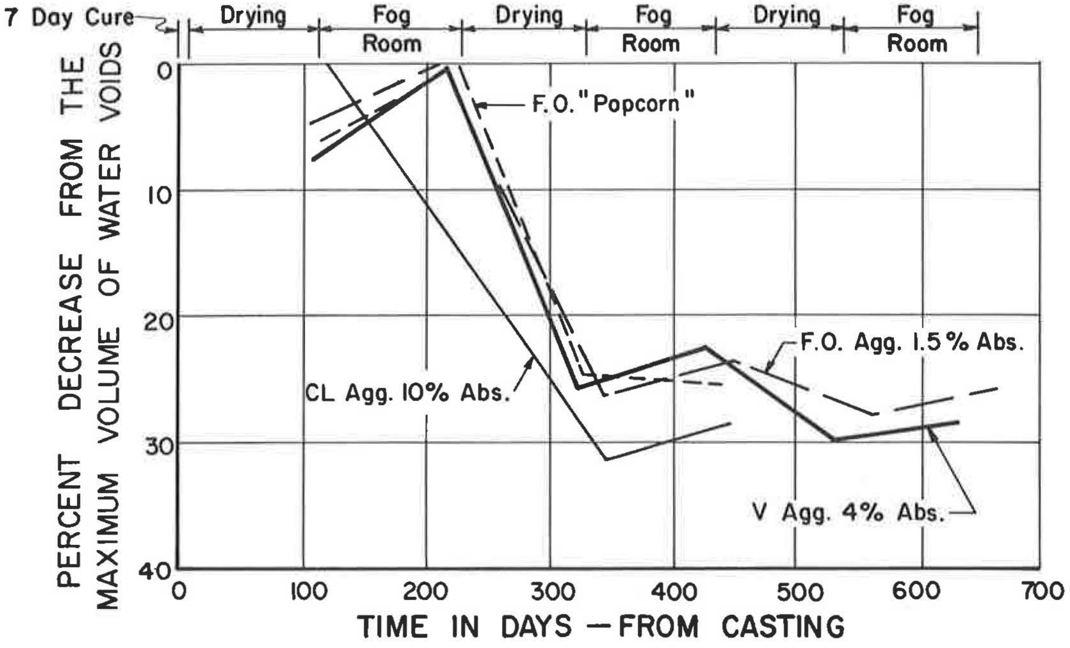


Figure 8. Influence of aggregate on volume change of water voids-cement content constant. Water voids computed from measured volume of moisture gained and lost during indicated exposure; concrete not oven dried; drying and fog room at 73.4 F, and 50 and 100 percent relative humidity, respectively.

on the basis of relative volume. Therefore, when a concrete block was in a saturated condition and lost a measured quantity of water, the volume of concrete that had contained this weight of water would be a function of the absorption of the concrete. Because the concrete blocks had constant dimensions, the depth below the surface of the block was computed, which would give the necessary volume of concrete that could hold the measured loss of water. Calculations of this type were used to determine the necessary depth below the concrete surface that would account for the measured gain and loss of water. It is readily apparent that this method of computation assumes that a portion of the concrete is relatively dry, whereas another portion may be saturated. The reality of this assumption is highly speculative, as would be other assumptions on the relative amount of moisture contained at various points within the concrete mass.

An analysis of the data indicated that there were two predominant variables which influenced the calculated rate of loss of moisture during the exposure in the dry room at 73.4 F and 50 percent relative humidity: (a) the cement content and (b) the assumed volume of the concrete that contained the evaporable water. A plot of these data is given by

$$t_y = 10^{0.0442C} S_i^{2.22} 0.086 \tag{3}$$

in which

- t_y = time in years to dry to equilibrium with an environment of 73.4 F and 50 percent relative humidity;
- C = sacks of cement per cubic yard of concrete; and
- S_i = depth below surface in inches.

Analysis of the data indicated that the absorptive characteristics of the aggregate per se were not a predominant influence on the calculated rate of moisture evaporation.

The calculated rate of moisture movement through the concrete when exposed to the fog room environment of 73.4 F and 100 percent relative humidity implied that the concrete mix variables were not primary test variables. For a calculated depth of approximately 75 percent of the half-depth of the concrete, all specimens absorbed moisture at a rate of about 0.2 in. per day, then reduced in velocity at greater depths.

The results of the absorption test of the concrete in the fog room appear to be inconclusive in determining the influence of the concrete variables on the calculated rate of absorption. Although the blocks were exposed in a fog room of controlled temperature and humidity, the fog dispersion within the room was not uniform. For this reason the concrete blocks were moved about the room to average out the influence of differences of fog dispersion. It is possible that the variations of the fog room environment are greater than the influence of the concrete mixes. The data indicate that the rate of moisture absorption of the concrete in this test was greater than evaporation rate. Therefore, the evaporative characteristics of the concrete appear to be the controlling variable in the transpiration of moisture through concrete.

Relation of Laboratory and Field Studies

Field exposure tests showed that the protection of reinforcing steel by concrete is a variable that depends on the type of cement, water-cement ratio, admixtures, and thickness of concrete cover over the reinforcement (9, 10). Thus far, this investigation has not directly considered the influence of concrete variables on preventing or inhibiting the corrosion of reinforcing steel. However, it is probable that the durability of reinforced concrete will be a function of at least two of the many variables: (a) the absorption, and (b) the rate of moisture movement through the concrete.

The test methods used in the study precluded direct measure of the relationship of the saturated surface-dry to oven-dry method for measuring concrete absorption of water. However, the quantity of absorbed water found in this study could be related to the amount determined in the oven-drying method.

The concrete variables observed were related to the performance of the concrete in an atmospheric environment. It is believed that the behavior of concrete in such an environment is similar to the performance of concrete exposed near the ground or water line. This assumption is based on the measured differentials in the chloride content of concrete that has been exposed to a marine environment. An earlier study (1) showed that the chloride content decreases with depth below the surface. Therefore, with a given chlorinity of the environment, the observed differences in salt content of the concrete are assumed to be caused by wetting and drying.

With other environmental conditions being equal, it appears that the quantity of chlorides deposited within various concrete mixes could be directly proportional to the rate at which water could move from within the concrete to the atmosphere. This could be given by

$$Q = T_y W_v \tag{4}$$

in which

Q = quantity of water evaporated per year;
T_y = rate or cycles of evaporation per year; and
W_v = volume of water evaporated per cycle.

As previously stated, Eq. 3 gives the time in years when concrete would dry to equilibrium to a calculated depth. To obtain the rate or cycles of drying per year, it is necessary to take the reciprocal of that equation; thus, the rate per year of drying is

$$T_y = \frac{1}{10^{0.0442C}\, S_i^{2.22}\, 0.086} \tag{5}$$

The volume of water as a percent of concrete volume that would be evaporated from mature concrete is given by Eq. 2. Combining Eq. 2 with Eq. 4 gives

$$Q_y = T_y\, V_w\, V \tag{6}$$

in which

Q_y = quantity of water evaporated per year in cubic inches;
T_y = rate or cycles of drying per year;
V_w = water voids as percent of concrete volume; and
V = volume of concrete in cubic inches.

Substitution of Eqs. 2 and 5 in Eq. 6 gives

$$Q_y = \frac{0.0988 W_m^{1.17} A}{10^{0.0442C}\, C^{0.717}\, S_i^{1.22}} \tag{7}$$

in which

Q_y = cubic inches of water evaporated per year at 50 percent relative humidity and 734 F; and
A = cross-sectional area in square inches.

Construction records indicate that the over-all average design of the concrete mix for reinforced concrete piles for the past 50 years could be $C = 6$ sacks of cement per cubic yard of concrete, $W_m = 22$ percent by volume, total mixing water, and $S_i = 2$ in. of concrete cover over the reinforcing steel. Using these basic figures, the yearly quantity of water evaporated from an average concrete pile that is saturated with water and exposed to 73.4 F and 50 percent relative humidity could be, using Eq. 7, 0.2356A.

Eq. 1 gave the total yearly percentage of structures found in the field to have corrosion of the reinforcing steel. On a design basis, an adequate level of confidence would be attained if it were known that 70 percent of the structures placed in similar environments had shown deterioration at a particular number of years of service. Therefore, by substituting 70 percent into Eq. 1,

$$R_a = \frac{424}{K^{0.42}} \tag{8}$$

in which R_a is the number of years when 70 percent of the structures constructed of average concrete and placed in an environment of K (chlorides in parts per million) would have visual evidence of corrosion of the reinforcing steel in the substructures.

The time for deterioration of a reinforced concrete substructure would probably be proportional to the rate of moisture evaporation of the concrete to the considered depth for various design variables. Therefore, an equation encompassing differences in the evaporative characteristics of an average and a specific concrete is

$$R_t = R_a \times \frac{Q_a}{Q_s} \tag{9}$$

in which

R_t = years to deterioration of bridge substructure;
R_a = years to deterioration of an average bridge substructure;
Q_a = quantity of water evaporated by average concrete; and
Q_s = quantity of water evaporated by specific concrete.

Substitution of Eqs. 7 and 8 in Eq. 9 gives,

$$R_t = \frac{10^{0.0442C}\, C^{0.717}\, S_i^{1.22}\, 1,011}{K^{0.42}\, W_m^{1.17}} \tag{10}$$

Figure 9 shows a solution of Eq. 10. Figure 10 shows how this equation calculated from the data for average construction practice fits the field conditions as observed for the inspected bridges in California.

Correlations to Independent Tests

There is a general lack of detailed data in the literature that would independently test the accuracy of the results of this investigation. One of the more detailed investigations that have been reported in sufficient scope is that by Lea and Watkins (9). Their test of the protective value of concrete in preventing the corrosion of reinforcing steel was started in 1929 at two locations in England. One exposure site was in natural sea water at Sheerness, and the other was at Watford in sea water concentrated to three times its normal salinity.

At each of the test sites, concrete piles of 5 in. square in section and 5 ft long were exposed to the two environments. The piles were constructed of mixes that varied from ½ to 2 in. of slump; 3.8, 6.4, and 10.6 sacks of cement per cubic yard; and 1 and 2 in. of cover over the reinforcing steel. Unfortunately, this test was not continued under the original test conditions beyond 10 years. However, comprehensive data are available for the first 10 years of controlled exposure and are compared to the estimated time to deterioration given in this study (Table 1).

In an attempt to correlate the empirical equation of California experience to the reported test data, a statistical comparison was made by the method of least squares for the piles exposed at Sheerness. The authors stated that the exposure conditions at Watford were not as severe as Sheerness; therefore, the statistical comparison was not made for this location. For six degrees of freedom, the level of significance was approximately 0.01 with a correlation coefficient of 0.809. The standard error of estimate was $\pm$ 1.92 years.

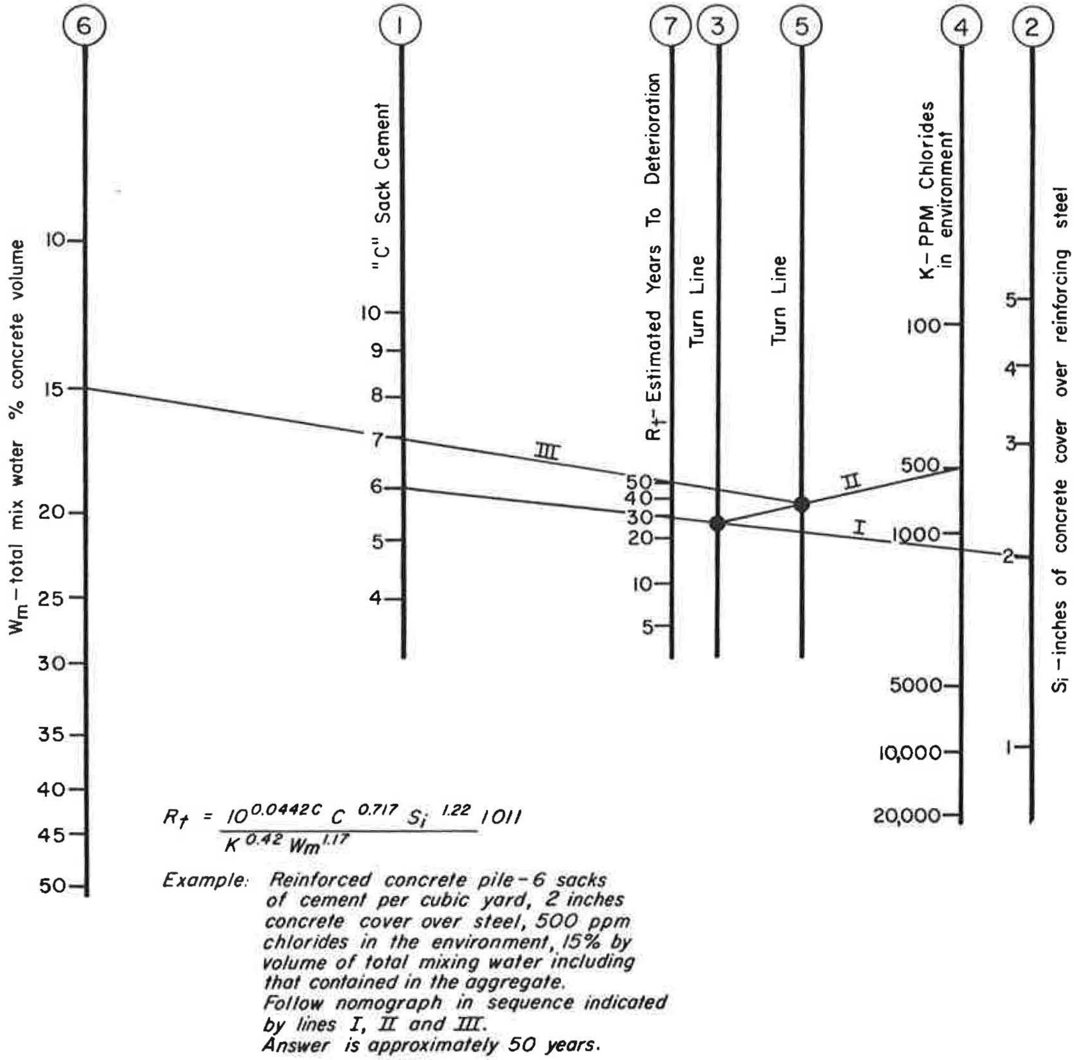


Figure 9. Chart for estimating deterioration time for reinforced concrete pile.

Due to the lack of details as to when the reinforced concrete test piles had cracked in the long-time study of cement performance (10), it is not possible to correlate those test results directly to the estimated performance results of this study. Even so, it was stated in the 10-year report "that for the salt water exposures the effect of rusting and expansion of the reinforcing steel is a major deteriorating influence." All the concrete piles were so constructed that there was 1.5 in. of concrete cover over the reinforcing steel. The concrete tested consisted of the following mixes: Mix 1, 5 sacks of cement per cu yd and 2 in. slump; Mix 2, 7 sacks, 2-in. slump; Mix 3, 7 sacks, 8-in. slump. The estimated time to corrosion of the reinforcing steel for the piles made of the reported concrete mixes was less than 8 years. At the St. Augustine test site, the data showed that after 15 years exposure, for Mix 1, 89 percent of the total piles were cracked; for Mix 3, 47 percent were cracked, and 16 percent of the Mix 2 piles were cracked. After about 15 years of exposure at the Corona Del Mar test site, 100 percent of the Mix 1 piles were cracked, whereas 86 percent of the Mix 3 and none of the Mix 2 piles were cracked.

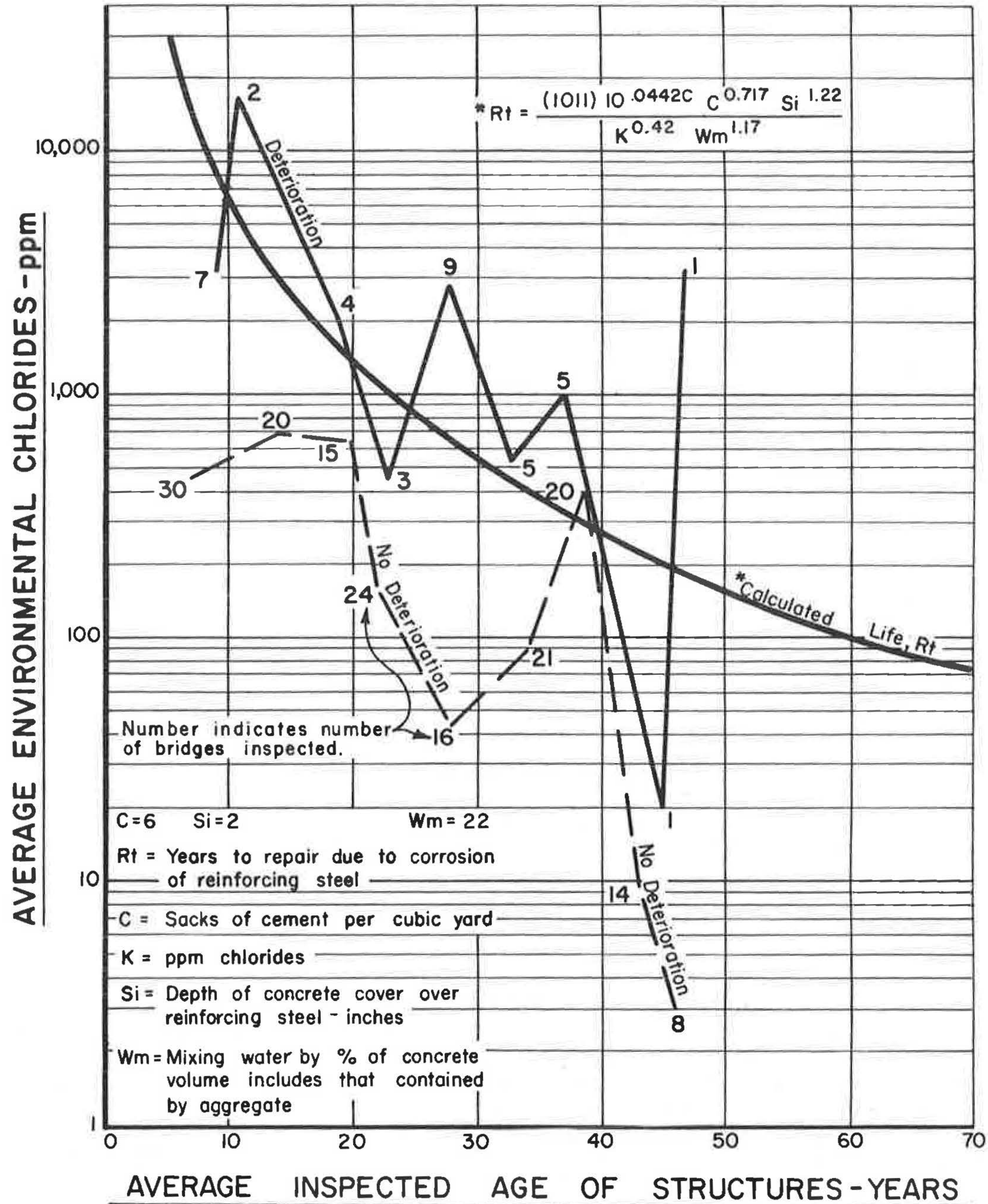


Figure 10. Condition of 205 bridges vs average age at inspection and chloride concentration.

TABLE 1

COMPARISON OF THEORETICAL AND ACTUAL TEST DATA[a] FOR REINFORCED CONCRETE PILES

(Sacks/Cement cu yd)	Concrete (in.)		Age at Cracking[b] (yr)			
			Sheerness		Watford	
	Slump	Cover	Actual	Estimated[c]	Actual	Estimated[c]
10.6	½	1	6	8	-[d]	5
		2	-[e]	20	-[d]	11
	2	1	10	8	-[e]	4
		2	-[e]	18	-[d]	10
6.4	½	1	3	4	1	3
		2	10	10	-[e]	6
	2	1	2	4	1	2
		2	10	9	-[e]	5
3.8	2	1	3	2	3	1
		2	8	5	6	3

[a]Data obtained from (9) as results of field exposure tests in England.
[b]Of 5-in. by 5-in. by 5-ft test pile.
[c]Based on Eq. 10.
[d]Test conditions altered after 10 years, cracked in 10 to 20 years.
[e]Test conditions altered after 10 years, not cracked in 20 years.

Although the actual time when a pile had cracked due to reinforcing steel corrosion was not reported, the deterioration of the piles apparently occurred at an early age, which confirms the corrosion time computations to a limited degree. However, the estimated time to deterioration as indicated by this study does not infer a degree or rate of deterioration at any location. The data indicate the time to the incidence of deterioration.

As a result of corrosion of the steel in reinforced concrete bridge piles in Texas, extensive repair was required after approximately 7.5 years of service (12). The original concrete mixture in this structure consisted of 5 sacks of cement per cu yd, 6.5 gal of water per sack of cement, and 2 in. of cover over the reinforcing steel.

The stream water was reported to be "loaded" with chlorides. The total quantity of chlorides was not reported, although deposits of salts were clearly visible on the surface of the banks of the stream. Assuming a concentration of chlorides equal to that found in sea water, which could be considered as "loaded" quantity, the estimated years of service to corrosion of a pile in this structure would be 8 years. Based on the assumption of the quantity of chlorides in the Texas bridge environment, the field data agree with the estimated life.

The results of an inspection of reinforced concrete structures on the east coast of the United States reported in 1957 (6) showed that 6 out of 10 structures had reinforcement corrosion at an average inspected age of 23 years. Although neither construction details nor the initial time when the structures were found to be deteriorated was reported, the inspection indicates that the normal construction methods, which were used until 1946, were not adequate to insure an indefinite maintenance-free service life of structures exposed to sea water. In sea water, the expected time to corrosion of piles based on a 6-sack mix, with 15 percent total water by volume and 2 in. of cover is 10 years.

ANALYSIS

From the viewpoint of an investigator, it has been extremely fortunate that the basic design of reinforced concrete substructures in California has not varied within drastic limits for the past 50 years. Otherwise, it would have been exceedingly difficult, or even impossible to use the data given in this paper to correlate the condition of the inspected structures with the environment. Even though a correlation was found, the data only reflect the normal exposure of bridge substructures for that portion which can be observed. Underwater or underground sections were not investigated, and the application of these data to those conditions should be tempered with performance data.

For the purpose of comparing variables in concrete design, a direct ratio (of the evaporative characteristics) was established between the average concrete used for 50 years and the various types exposed to the laboratory test conditions. It is speculated that this relationship of relative concrete evaporative characteristics would be consistent with that observed in other environments. On this basis, care should be used in applying these data to structures not directly exposed to the free flow of air, such as that found in highway bridges. For instance, 70 percent of the highway bridges built with the recommendations of concrete design by some California harbor departments (13) and consequently exposed to sea water would be expected to have corrosion of reinforcing steel after approximately 13 years of service. Evidently, there must be an environmental difference in the exposure conditions between a highway bridge and harbor facilities on the California coast, and between various harbor departments. For instance, Gaye and Agatz (14), commenting on the construction of dock works in Germany during the 1920's, indicated that 10 cm of covering is necessary to protect the iron elements from corrosion. Using their described concrete mixtures, a California bridge would probably show deterioration of the substructure in approximately 20 years. However, these German structures were built of concrete to which trass had been added to the mix. English tests (9) have shown that, for comparable conditions, concrete made of trass cements had proved better than portland cement for inhibiting the corrosion of reinforcement. Also, results of the tests performed in the United States indicate that the rate of deterioration of reinforced concrete piles varies not only with the basic mixes used but also with the type of cement (10).

There appear to be many factors (6, 8, 15) that could influence the deterioration of a structure exposed to a corrosive environment, such as the influence of the type of cement (6, 8, 9, 10, 13); variables of aggregate size, grading, or manufacture (13); additives (9, 14, 15, 16, 17, 18); workmanship (2, 6, 8, 13, 14, 18); curing (6); and the environment (15, 19, 20). These factors require considerable attention when designing a structure for a specific maintenance-free service life.

SUMMARY

A total of 239 reinforced concrete bridges were inspected throughout the State of California, and it was found that corrosion was attacking the embedded steel in bridges that were in widely dispersed geographic locations. A study of the data indicated that the primary cause of corrosion was the presence of chlorides in the environment. Apparently, the chlorides permeate the concrete, and, after a period of exposure, depending on chloride concentration in the natural soils or waters, bring about the corrosion of the reinforcing steel.

Based on mathematical distribution curves, it was found that 70 percent of the structures placed in an environment of a certain chloride concentration would have corrosion of the reinforcement according to Eq. 8. By the method of least squares, it was found that this formula duplicated the distribution curve of the field results within about 7 percent of the actual percentage of structures affected by corrosion.

The past history of bridge construction in the State of California indicates that the average reinforced concrete piles have generally been made of a mixture of 6 sacks of cement per cubic yard of concrete and 2 in. of concrete covering over the steel.

A mathematical analysis of the data indicates that the concentration of sulfates in the environment is not the primary cause of reinforcing steel corrosion. It is speculated that the presence of excessive sulfates could lead to the corrosion of reinforcing after it had caused chemical attack and possibly disintegration of the concrete.

Laboratory tests were made of the moisture absorption and evaporative characteristics of various concrete mixes consisting of combinations of the following: 4, 6, 8 sacks per cubic yard; 2, 4, and 6 in. of slump; aggregate of 1.5, 4, and 10 percent absorption by weight; and various mixtures of neat cement bars. When these test specimens were exposed in a fog room at 73.4 F and 100 percent relative humidity, the calculated rate moisture absorption of the various mixes could not be distinguished from each set of specimens. Evidently, the variables of the fog environment were such that the small differences in the absorption rates of the concrete mixes could not be detected.

When the concrete specimens were exposed to the dry room at 73.4 F and 50 percent relative humidity, it was found that the rate of moisture evaporation at the calculated depth would vary according to Eq. 7.

Because of the relationship of the chloride concentration in the environment to the deterioration time of the structures, it was assumed that the time to the build-up of a harmful quantity of salts was related to the volume of water that could evaporate in the various mixes and thus would deposit an accumulating quantity of chlorides in the concrete. Eq. 7, which gives the relationship of the various mixes to the rate of moisture evaporation, was used with Eq. 9 to give Eq. 10, the basic equation for estimating the probable number of years to corrosion of the reinforcing steel in 70 percent of the structures placed in the normal highway bridge environment.

This derived equation (Eq. 10) does not imply that a structure will be in structural distress at the indicated time. It only represents the number of years that could be expected to elapse before corrosion of the embedded steel could cause rust stains or cracking of the concrete. The equation does not indicate the rate at which corrosion will occur in different substructure members of the same bridge, only when the first member (pile) will be visibly affected. It is expected that the longer the period to the first evidence of corrosion, the greater the time to the visible distress of the last member of the same structure.

Correlating these data to those reported by others was difficult because of a lack of construction or environmental details. However, in the exposure tests of piles in England, the indicated age to distress as determined by this test method was within approximately 2 years of the actual time required for distress for the 10 years of controlled environmental conditions. Other comparisons were made between the reported conditions of structures elsewhere in the United States, and although the test could not be related in terms of years, it did indicate whether a designer could anticipate the premature deterioration of the bridge substructure.

The work presented in this paper should not be thought of as the "final answer." Instead, it is hoped that a step has been made toward finding a means to evaluate the effect of the environment on the durability of reinforced concrete bridge piles, and that attention has been directed to the kinds of data needed to predict service life.

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The Value of Insulated Forms for Winter Bridge Construction

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• SUFFICIENT literature is available to support the theory that winter-produced concrete can be adequately protected in its initial stage by the use of insulated forms. The purpose of this paper is to report the findings of the New York State Department of Public Works in its use of insulated forms for winter concreting on bridge construction for the period from 1957 to 1960.

Scurr (1) of the South Dakota State Highway Commission must be recognized for his responsible reporting of work done in this field at a time when the need was great and the answers were few. As with all original work, certain refinements may be made by others whose interests are only to add to the storehouse of knowledge by using the basic thesis objectively in gathering additional data.

In general, the temperature in New York State can be expected to be below 32 F for an average of 125 days a year. In South Dakota, the temperature can be expected to be below 32 F for an average of 163 days a year (Fig. 1).

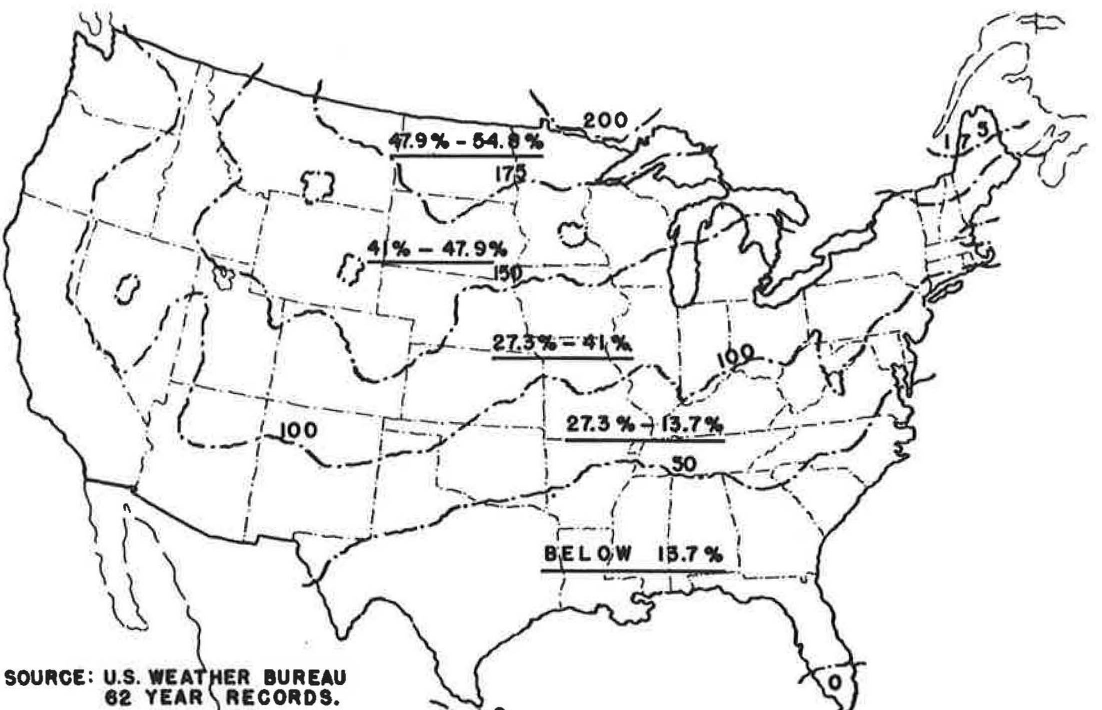

Figure 1. Day contour map, number of days temperature is below 32 F.

Paper sponsored by Committee on Construction Practices— Structures.

From the relationship of these two States it could be expected that the ambient air temperatures of South Dakota would be somewhat lower than those encountered in New York State. This could, in part, be responsible for the difference in the results reported by Scurr and those obtained in New York State.

The pros and cons of the required protection for winter-produced concrete are amply covered in Scurr's paper (1). One observation, however, might be made regarding the great amount of construction carried on in the metropolitan areas of New York State. In many instances the only space available to the contractor is actually the construction limits of his contract. The employment of housing and salamanders or heating elements would deny the contractor freedom of movement in his already confined area. The use of insulated forms releases to the contractor this much needed usable space.

New York State instituted its first program of using insulated forms for the protection of winter-produced concrete during the winter construction season of 1956-1957. This program was carried out on three contracts all in the Buffalo area. The specifications for control were limited, in that it was stipulated that 2-in. insulation be used, but in all other respects the concrete should be formulated in compliance with the current specifications for concreting in cold weather.

The pertinent requirements contained in the construction specifications for concreting in cold weather were predicated on the use of external heat:

> All water used for mixing concrete shall be heated to a temperature of at least 70 F but not over 150 F. Aggregates shall be heated either by steam or by dry heat to a temperature of at least 70 F but not over 150 F. The heating apparatus shall be such as to heat the mass uniformly and preclude the possibility of the occurrence of hot spots which will overheat the material.
>
> The temperature of the mixed concrete shall be not less than 60 F at the time of placing in the forms.
>
> In cases of extreme weather conditions the Engineer may, at his discretion, raise the lower limiting temperatures for water, aggregate and mixed concrete.

Figures 2 and 3 show the method of applying the insulating blankets. Figure 4 shows the location of thermometers and thermocouples.

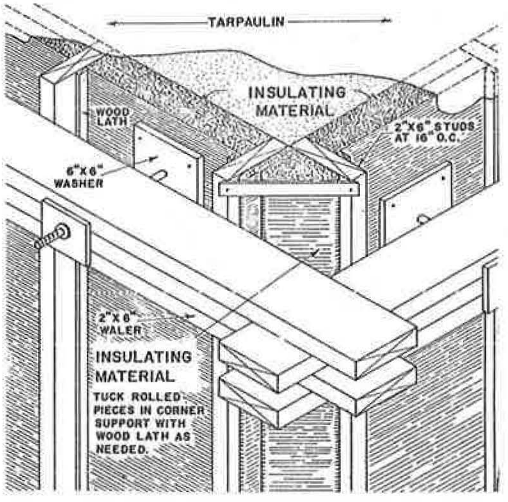

Figure 2. Method of applying insulating blankets, wood forms.

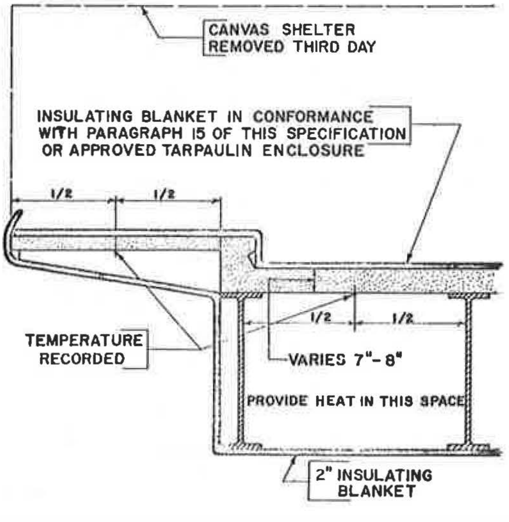

Figure 3. Method of applying insulating blankets, steel forms.

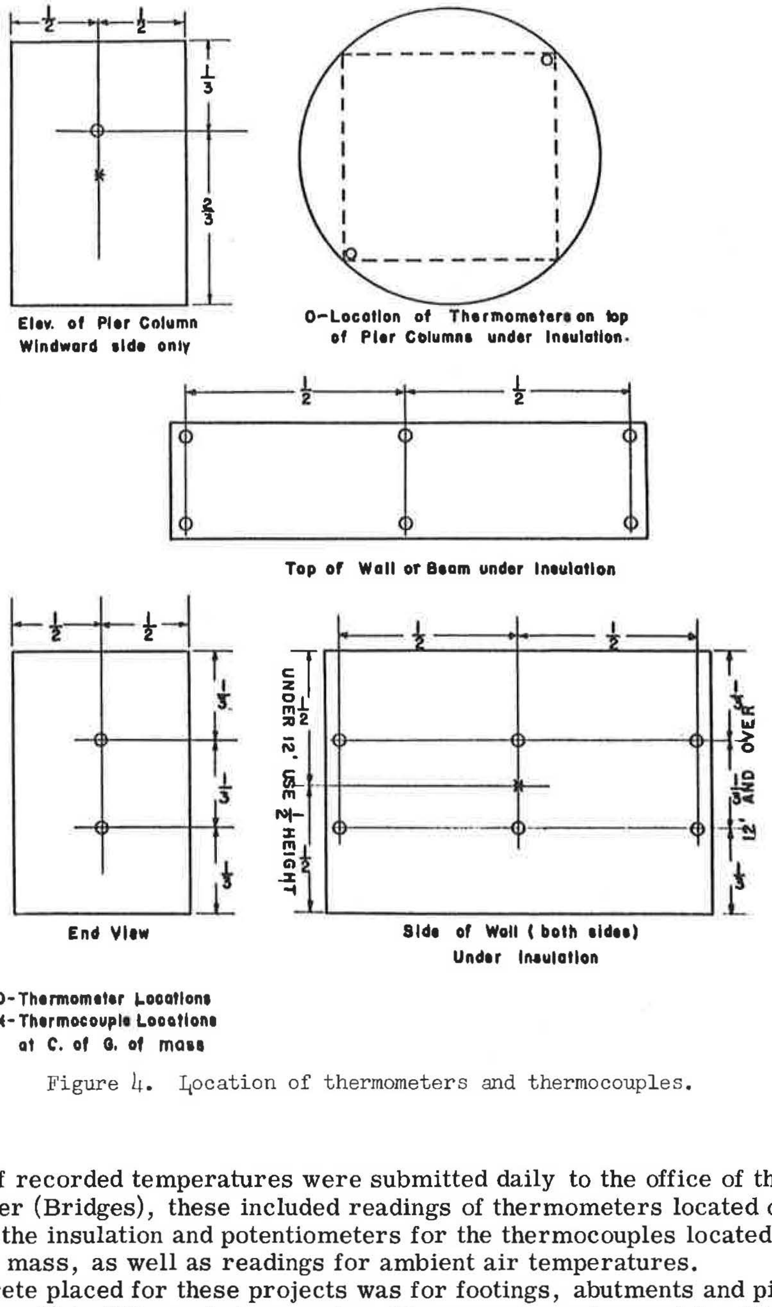

Figure 4. Location of thermometers and thermocouples.

Reports of recorded temperatures were submitted daily to the office of the Deputy Chief Engineer (Bridges), these included readings of thermometers located on the forms under the insulation and potentiometers for the thermocouples located at the center of the mass, as well as readings for ambient air temperatures.

The concrete placed for these projects was for footings, abutments and pier beams requiring from 50 to 130 cu yd of concrete. The cement requirement for this concrete was 6 bags per cu yd. The forms used were ¾-in. plywood, and the temperature of the concrete at time of placement was 75 F. The ambient air temperature at time of placing the concrete was 44 F and the average ambient air temperature for the 5 days of curing was 36 F.

Internal temperatures were reported as high as 158 F at approximately 32 hr after placement. Temperatures on the side of forms under the insulation were reported as high as 129 F at approximately 32 hr after placement. Temperatures on top of concrete under straw were reported as high as 98 F at approximately 24 hr after placement.

In an attempt to counteract this high internal temperature, the engineers in the field were instructed to crack the forms after the second day. This proved expedient though at the same time a doubtful way to control temperatures of concrete when using this type of construction.

Concrete developing the temperatures indicated would be vulnerable to thermal shock should the forms be removed under normal construction procedures. There is also a possibility that the excessive internal temperatures could produce varying expansions and, on cooling, cause internal fractures that could destroy the homogeneity of the mass. This expansion could possibly result in a variation of the elevation of the top surfaces of the unit from that indicated on the plans.

During the summer of 1957 an intensive study was made on this subject, and in the fall of 1957 an alternate specification for concreting in cold weather was issued together with a letter of instruction, to the engineers in charge of projects. This specification, amended, and the letter of instruction are given in the Appendix.

The specification required that concrete units having a depth of 24 in. or less would require the 2-in. insulation on the forms, the concrete temperatures at placement should be not less than 50 F nor more than 60 F. Concrete sections having a depth greater than 24 in. would require 1-in. insulation on the forms and the temperature of the concrete at placement should be not less than 40 F nor more than 50 F. This specification also included a table of insulation requirements for concrete walls, piers, abutment, and floor slabs above ground. The data contained in that table can be found elsewhere (2, 3). There is, however, no reference made to indicate whether the form material is wood or steel. It has been determined that these data were predicated on the use of steel forms that have no insulating value. Because this fact has been established, it is possible to reduce the thickness of the insulation on wood forms.

During the 1957-1958 winter construction season, 40,000 cu yd of concrete were placed using insulated forms. Results obtained were significantly better than those of the 1956-1957 winter construction season. The greatest difficulty encountered was impressing on all parties the necessity of reducing the temperature of the concrete at time of placement.

Figure 5 shows the resulting temperatures for a pier beam. This unit was protected with 2 in. of insulation on wood forms. Temperature of concrete at time of placement was 74 F. Internal temperatures of 140 F were recorded 28 hr after placement. Temperatures of 112 F were recorded on sides of forms under insulation 48 hr after placement. Temperatures of 96 F were recorded on top of unit 52 hr after placement. The ambient air temperature at time concrete was placed was 50 F and did not go below 32 F until early on the fourth day. Internal heat loss started approximately 54 hr after placement at an average rate of 4 F per day for 3 days. On the last recorded day the internal heat loss was 24 F. Heat loss indicated from report of readings on sides averaged 9 F per day. Heat loss indicated from report of readings on top averaged 8 F per day. During this same period the ambient air temperature had dropped 20 F at a rate of approximately 4 F per day.

Figure 6 shows the resulting temperature for a wall stem. This unit was protected with 1 in. of insulation on wood forms. The temperature of the concrete at time of placement was 70 F. Internal temperatures of 112 F were recorded 48 hr after placement. Heat loss averaged 9 F per day. Temperature of 74 F was recorded on side of forms 24 hr after placement. There was a temperature loss of 19 F after 4 days in place followed by a rise in temperature of 14 F at the end of 6 days. Temperatures of 68 F were recorded on top of unit 48 hr after placement. Heat loss was 14 F in the next 24 hr, then retaining a constant temperature for the remaining 3 days. In this instance, the ambient air temperature was 40 F at the time the concrete was placed and at no time did it register below 36 F. The ambient air temperature rose an average of 10 F per day for the final 2 days or to 55 F. However, the differential in temperature from the ambient air to the internal is but 15 F at the end of the 6-day period.

When placement temperature of the concrete was reduced, there was a noticeable reduction in peak temperature. From the records this peak temperature was not attained until 72 hr after the concrete was placed.

Figure 5. Pier beam temperatures.

Figure 6. Wall stem temperatures.

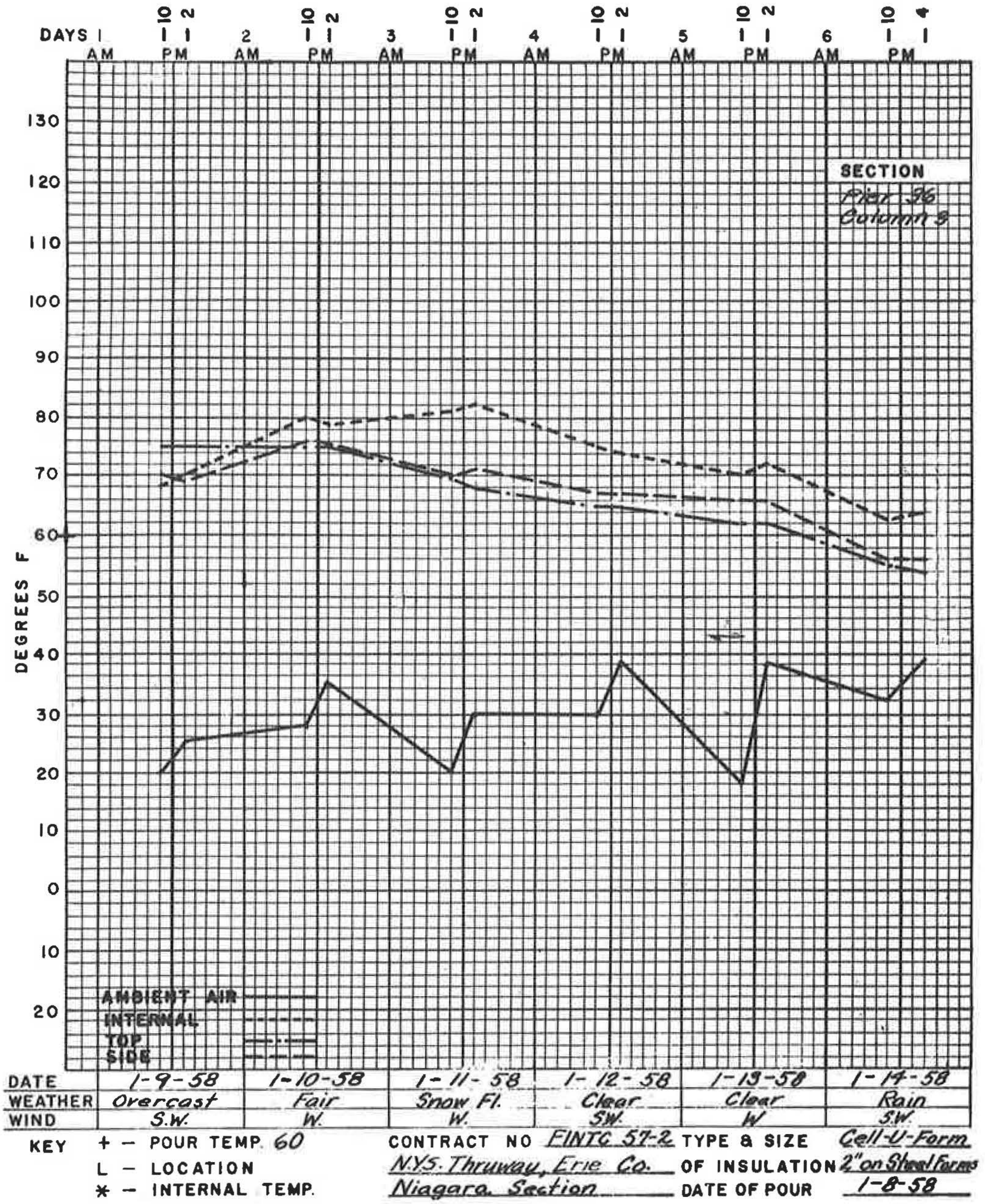

Figure 7. Heat evolvement from concrete placed at 60 F, with 2-in. insulation on steel forms.

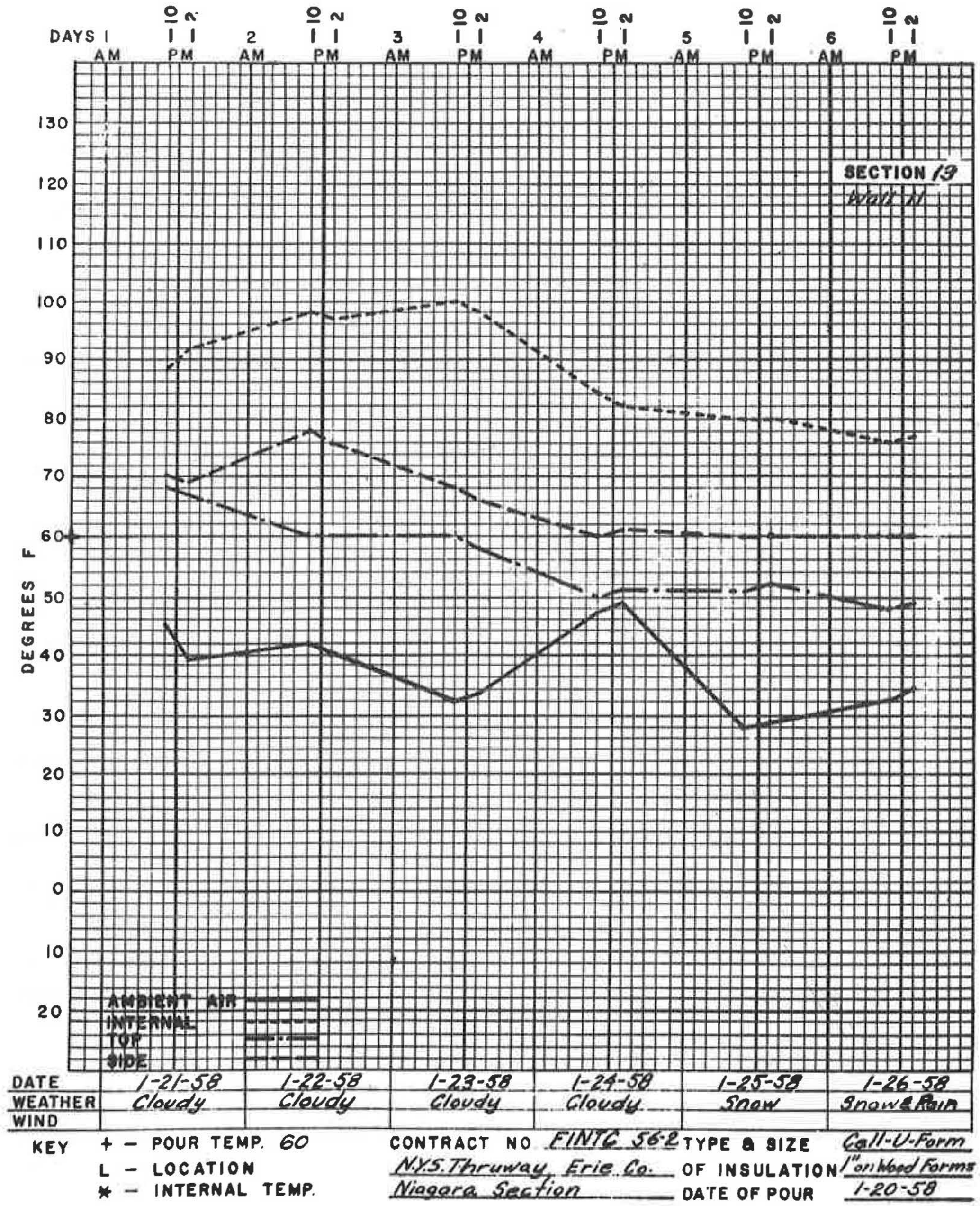

Figure 8. Heat evolvement from concrete placed at 60 F, with 1-in. insulation on wood forms.

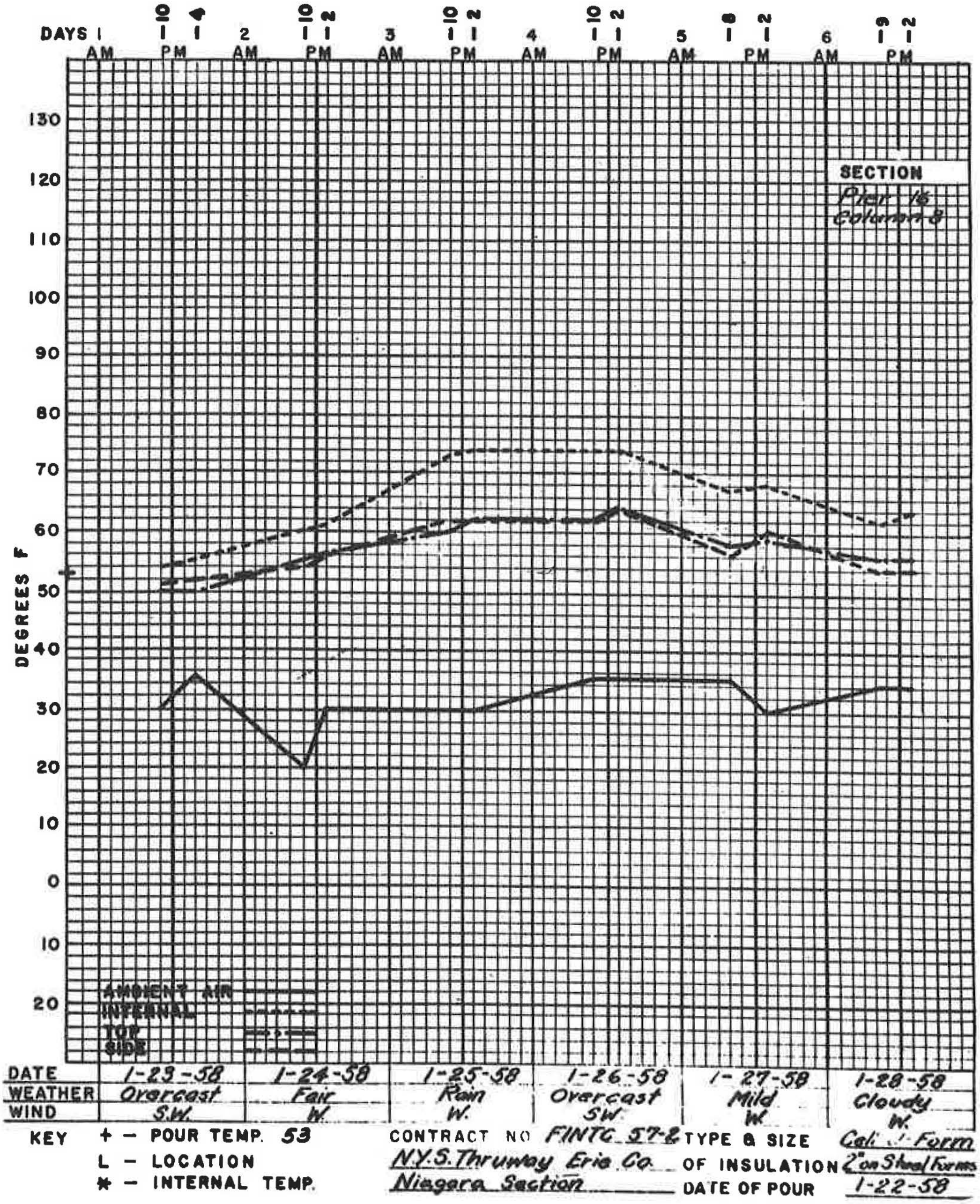

Figure 9. Heat evolvement from concrete placed at about 50 F, with 2-in. insulation on steel forms.

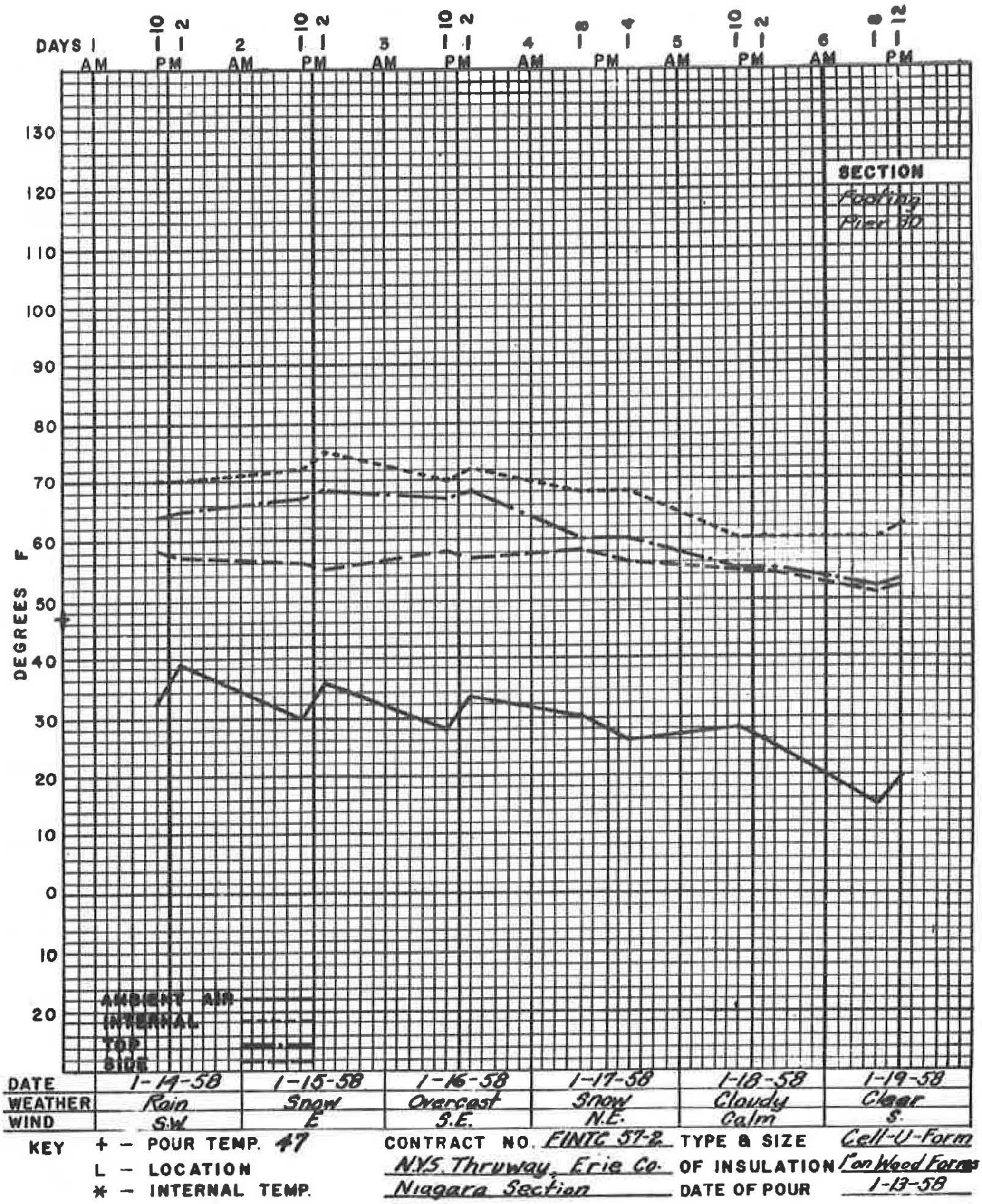

Figure 10. Heat evolvement from concrete placed at about 50 F, with 1-in. insulation on wood forms.

Figures 7 and 8 show heat evolvement when concrete is placed at 60 F. Figure 7 shows the use of 2-in. insulation on steel forms, and Figure 8 shows the use of 1-in. insulation on wood forms. Ambient air temperatures for these two conditions have an entirely different pattern, and it must be assumed that this, in part, is responsible for the temperature gradients attained for the concrete within the forms.

Figures 9 and 10 show heat evolvement when concrete is placed at approximately 50 F. Figure 9 shows the use of 2-in. insulation on steel forms, and Figure 10 shows the use of 1-in. insulation on wood forms. The average ambient air temperature for these two conditions was almost identical, and the temperature gradient patterns of the internal, top and side of the concrete within the forms were closely grouped.

The information contained in Figures 5 through 10 though specific in nature were chosen from some 1,500 reports as representative of the results obtained in four winter construction seasons. Test cylinder results on this concrete indicated an average strength of 5,200 psi.

With the many variables involved, it is difficult to resolve the requirements that satisfy all conditions encountered at time of construction. It is therefore necessary to draft a specification that will allow the engineer to interpret the requirements intelligently so that the best end results can be obtained.

Insulating materials and their coatings have been improved since the time of Scurr's initial work in this field. These improvements are indicated in the attached copies of the New York State Specifications.

Some 100 contracts will be progressed through the winter construction season of 1962-1963 by New York State, using insulated forms to protect the concrete produced. This has become a normal construction practice which has allowed the State of New York to keep current with the ever-expanding highway program.

ACKNOWLEDGMENTS

The writer is indebted to Gordon A. Erickson of the Wood Conversion Company for his technical assistance throughout the entire investigational period, to all the engineers and contractors for their cooperation and collection of all data pertinent to this program, and to William Scrom of the New York State Department of Public Works (Bridges) for compiling the reported data and preparing the graphs.

REFERENCES

1. Scurr, K. R., "Insulated Forms for Winter Construction of Bridges." HRB Bull. 162, 13-19 (1957).
2. Wallace, G. B., "Insulation Facilitates Winter Concreting." Bureau of Reclamation, Engineering Monograph 22.
3. "Recommended Practice for Winter Concreting." American Concrete Institute, ACI 604 (1956).

Appendix

ALTERNATE SPECIFICATION FOR CONCRETING IN COLD WEATHER

The following specification shall become a part of the contract document whenever concrete construction work is done under outside temperatures at below 40 F.

Protection of concrete by use of insulated forms or insulation laid on horizontal slabs is permitted under the following conditions:

<u>Product</u>

1. The insulating blanket shall be Cell-U-Forms as manufactured by Wood Conversion Company, Saint Paul, Minnesota, or approved equal.

The approval of any product as equal shall be determined by the Deputy Chief Engineer (Bridges).

2. The thermal conductivity (k) of the insulating blanket shall not exceed .27 Btu per sq. ft. per hour, per degree F. temperature difference between the two surfaces.

3. a) The 1" and 2" insulating blankets used on vertical forms shall have, on the side exposed to the weather, a 90 lb. double creped kraft liner or a polyethylene plastic liner, minimum thickness .004" or approved equal. The ⅝" insulating blanket shall have a minimum 50 lb. asphalt coated and creped kraft liner or a polyethylene plastic liner, minimum thickness .004" or approved equal. The interior liner completing the total enclosure of the blankets shall be a minimum 40 lb. kraft liner. The liners shall be asphalt bonded to both sides of the insulating mat. The liners shall have an extension on each side to form a flange so the blanket can be applied to the frame work of the forms.

3. b) Insulation used on horizontal concrete slabs shall have a polyethylene plastic liner, minimum thickness .004" or approved equal on both faces. The liner shall be asphalt bonded to both sides of the insulating mat. All edges shall be sealed.

Application of Insulation

4. a) The blanket insulation shall be applied tight against the wood form with the nailing flanges extending out from the blanket so they can be stapled or battened to the sides of the framing, which are either horizontal or vertical and spaced 12" to 16" o. c. The ends of the blanket shall also be sealed by removing a portion of the mat in order to bring the lines together and stapled or battened down to headers so as to exclude air and moisture. The corners or angles should be well insulated and held in place. See Figure 2.

4. b) In the case of steel forms the insulating blanket shall be applied tight against the form and held in an approved secure manner with ends sealed to exclude air and moisture.

5. Where practicable, the insulation (or insulated form) shall overlay any previously placed (cold) concrete by at least one foot.

6. Insulation of slabs on steel members shall be as indicated in Figure 3.

7. Any tears in the liner shall be patched or covered with a tacky waterproof tape or a piece of vapor barrier asphalted in place.

8. Where tie rods extend through the insulated form a plywood washer (¾" × 6" × 6" approx.) shall be placed on top of the insulation blanket and secured in a manner to provide adequate protection satisfactory to the Engineer.

9. The tops of all pours (horizontal and vertical) shall be covered with insulating blanket (Sect. 15) except for inaccessible areas around protruding reinforcement bars which may be insulated with salt hay or wrapped with approved insulation. Tarpaulins shall be used to cover the top of such pours as directed by the Engineer.

General Requirements

10. Outside temperatures at which concrete walls, piers, abutments or floor slabs above ground may be protected with insulation under various conditions as shown in Table 1.

11. a)Wood forms shall be insulated with an approved ⅝" or 1" insulation blanket as directed by the Engineer, on all sections more than 24" in thickness when concrete mixture is made with 5-½ bags or more of cement per cubic yard. On sections more than 48" in thickness a ⅝" insulating blanket shall be used. All sections 24" or less in thickness shall have forms insulated with an approved 2" insulation blanket.

11. b)Steel forms shall have an approved 1" insulation blanket on all sections more than 24" in thickness when concrete mixed is made with 5-½ bags or more of cement per cubic yard. All sections 24" or less in thickness shall have steel forms insulated with an approved 2" insulation blanket.

11. c)Insulation used to protect top of slabs shall be approved types of blankets 1", 1-½" or 2" as directed by the Engineer.

12. When insulated forms are used the temperature of the concrete mixture (5½ bags of cement or more per cu. yd.) shall unless otherwise directed have a temperature of not less than 50 F. or more than 60 F. for concrete sections having a thickness dimension of 24" or less, and not less than 40 F. or more than 50 F. in sections greater than 24" in thickness. When concrete is to be placed in contact with previously placed

(cold) concrete or in contact with an excessive amount of cold reinforcing bars or other steel members, the temperature inside of the insulated form shall be raised, as directed by the Engineer, to bring the temperature of the steel members to approximately 50 F.

TABLE 1

INSULATION REQUIREMENTS FOR CONCRETE WALLS, PIERS, ABUTMENTS AND FLOOR SLABS ABOVE GROUND

Cement Content lb. per cu. yd.	Wall Thickness (feet)	Minimum Allowable Air Temperature (Degree F.) for 1" Insul. Blkt.*	for 2" Insul. Blkt.*
400	.5	38	21
	1.0	22	-11
	1.5	8	-39
	2.0	2	-53
	3.0	- 6	
	4.0	- 8	
	5.0	-10	
500	.5	35	14
	1.0	15	-26
	1.5	- 3	-65
	2.0	-10	
	3.0	-20	
	4.0	-23	
	5.0	-25	
600	.5	32	6
	1.0	8	-41
	1.5	-14	-89
	2.0	-22	
	3.0	-34	
	4.0	-38	
	5.0	-40	

*The table is calculated for the stated thickness of blanket type insulation with a thermal conductivity of 0.25 Btu per hour per sq. ft. for a thermal gradient of 1 degree F. per in. Thermal resistance of the forms was not included, therefore, temperatures more accurately apply to concrete in steel forms. Use of wood forms would permit somewhat lower outside temperatures.

13. The Contractor shall provide thermometers to check the temperatures of the concrete as indicated in Figure 4.

14. Unless otherwise directed by the Engineer, the forms shall not be removed when the outside air temperature is 0 F. or below or when the weather forecast for the next 24 hours is for a temperature of 0 F. or below. They may be removed when outside temperature is below 32 F. providing the temperature difference between the air and the concrete surface is no more than 30 F. If possible forms shall be removed about the middle of the day to take advantage of the generally higher afternoon temperatures and radiant heat from the sun. However, in no case shall the forms be removed before the end of six (6) full days after the final placement of concrete in an individual unit.

15. Stand-by heat shall be provided if ordered by the Engineer. The application of exterior heat will only be necessary when thin slabs formed on one side only and in contact with structural steel are protected as indicated in Figure 3 or where the Engineer may direct in heavily reinforced sections, in lieu of altering the temperature of the concrete mix. When slabs are constructed and protected in conformance with Figure 3 external heat shall be introduced through the ducts formed by the enclosure, between the structural steel members prior to pouring the concrete, in order to preheat those members as directed by the Engineer. A uniform temperature of 50 F - 70 F. shall be maintained during the curing period. Transfer the external heat to the enclosure above the slab before the placement of the concrete, as directed by the Engi-

neer. The canvas enclosure shall provide protection for the slab and personnel during placing and finishing operation and shall be removed at the direction of the Engineer. Insulating blankets shall be placed on the surface of the slab as soon as the concrete has set so the surface will not be marred. The blankets shall be tightly butted together with the top of joints covered and edges held down with planks to prevent the wind from penetrating under it.

16. Permission to use this method does not relieve the Contractor of any of his responsibility under the contract nor does it change or modify any of the requirements of the specifications regarding concreting in cold weather except as stated herein.

ADDENDUM

Approved products for form insulation are:

1.) Celluform - as manufactured by Wood Conversion Company, St. Paul, Minn.

2.) Fiber glass curing blanket with fiber glass AF-15 filler - as manufactured by Owens-Corning Fiberglas Corp., Toledo, Ohio.

3.) Thermo-form and Thermo-slab - as manufactured by the National Gypsum Company, Buffalo, N. Y.

4.) Ultralite Glass Fiber Concrete Curing Blanket - as manufactured by Gustin-Bacon Manufacturing Co., 230 Park Ave., New York 17, N. Y.

5.) Ma Ka "Locked In" Insulated Blanket - as manufactured by Max Katz Bag Co., Inc., 316 So. New Jersey St., Indianapolis 4, Indiana.

October 22, 1958

Items 4 and 5 added
January 30, 1963

LETTER OF INSTRUCTION

Implementing Alternate Specification for Concreting in Cold Weather

TEMPERATURE OF CONCRETE PLACED - HEATING OF MATERIALS

The temperature of concrete when placed shall not be less than shown in the Alternate Specification for the class and type of construction indicated. Nor shall the temperature of the concrete when placed exceed that called for. The importance of proper pour temperature cannot be overemphasized. Concrete placed at low temperatures above freezing develops higher ultimate strength and greater durability than concrete placed at higher temperatures. High temperatures of freshly mixed concrete are always objectionable. Furthermore, higher temperatures require more mixing water, cause slump loss and sometimes quick setting, and increase thermal shrinkage. Rapid moisture loss from hot exposed concrete surfaces may cause plastic shrinkage cracks.

For air temperatures above 30 F with aggregates which are free of ice and frozen lumps, the desired temperatures of concrete can be obtained by heating the mixing water only. For air temperatures below 30 F, it is usually necessary to heat the aggregates to no more than temperatures of 35 to 40 F when the temperature of the mixing water is at 140 F. If the rock is dry and free of ice and lumps, adequate temperatures of fresh concrete can be obtained by only increasing the temperature of the sand which will seldom have to be higher than 100 F when the mixing water is 140 F.

Mixing water should be heated under such control and in sufficient quantity to avoid appreciable fluctuations from batch to batch. To avoid the possibility of flash set when water is heated to a temperature in excess of 100 F, water and aggregate should come together first in the mixer in such a way that the high temperature of one or the other is reduced before cement is admitted. Loss of potency of air-entraining agent is a possibility in the presence of hot water. Therefore, the agent must be placed in the batch after water temperature has been reduced.

TEMPERATURE RECORDS

Inspectors shall keep a record of the date, hour, outside air temperature (AM and PM), weather (calm, windy, clear, cloudy, etc.) and wind velocity and direction. The record shall include temperatures at all points indicated in Figure 4 of the Alternate Specification. It shall also include temperatures of the aggregates and water, the temperature of the concrete when placed, and thermometer reading shall be recorded giving the hour and dates of the reading for a minimum period of six days, with two readings per day, one in the morning and one in the afternoon.

The type of mix used (1-2-3½, 1-2-4) shall also be indicated.

TEST CYLINDERS

The test cylinders for this type of construction shall be job-cured consistent with the curing of the concrete in the structure insofar as it is practicable.

SELECTION OF INSULATION BLANKET - WOOD FORMS

When using wood forms for pours between 24" and 48" in thickness, the Engineer should base his selection of ⅝" or 1" insulation on the changeable climatic conditions consistent with the geographical location involved.

October 23, 1958