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Contents

Ice Removal on Highways and Outdoor Storage of Chloride Salts

BLAINE F. HIMMELMAN, Assistant Research Engineer, Minnesota Department of Highways

This study was undertaken to determine the comparative effectiveness of chloride salts and abrasive-chloride salt mixtures for ice removal, and also to study the storage characteristics of sodium chloride, calcium chloride, and mixtures of these materials. Controlled, field ice-removal tests were run on 15 combinations of materials within three temperature ranges. Data were collected on thickness of ice, actual quantity and location of chemical or abrasive applied, and periodic condition of the ice with regard to amount of ice removed. The outdoor storage characteristics of seven bulk materials and one packaged material under a light polyethylene sheeting were studied. The materials were sampled for moisture, crusting, and caking for a period of ten months.

A mixture of $\frac{1}{3}$ $CaCl_2$ and $\frac{2}{3}$ NaCl appears to be one of the better economical materials for ice removal, and this mixture was found to store well in bulk for a period in excess of ten months. Straight NaCl was found ineffective in clearing a wheelpath within a 60-min time period as compared with the mixture of $\frac{1}{3}$ $CaCl_2$ and $\frac{2}{3}$ NaCl. Also, NaCl cannot be stored outdoors in bulk longer than two months without considerable caking. Mixtures of salt, with abrasive were found to be relatively ineffective for removing ice below about 15 F. Further research on similar and additional chemicals should be conducted under well-controlled field conditions.

• THE INCREASED daily use of highways throughout the country has brought on a demand for safer and more efficient roadways for use during every season of the year. One of the largest problems confronting engineers charged with the maintenance of these streets and highways in the northern areas is the efficient removal of accumulated ice and snow.

Years ago, efforts to make roadways safe for motor vehicle travel consisted of the use of mechanical equipment to plow the roadways reasonably free of snow and the use of abrasives to provide some degree of traction on hills and curves. These efforts, however, were confined chiefly to the primary routes. In later years, the use of abrasives became more widespread and it was found that through the use of a mixture of calcium chloride and an abrasive the freezing of the abrasive stockpiles was eliminated. Secondly, it was found the chemical also aided in the embedment of the abrasive material in the ice or compacted snow.

The use of straight calcium chloride and sodium chloride was begun on a limited scale in the late 1920's and early 1930's. Because of an apparent detrimental effect to portland cement concrete pavement containing no entrained air, the use of straight chemicals was not recommended for general use (1) but was recommended where severe conditions existed. In areas where straight chemicals were found necessary it was advisable to plow off the resulting slush to minimize the damage.

Paper sponsored by Committee on Snow and Ice Control.

More recently, the use of straight calcium chloride, sodium chloride, and mixtures of these chemicals has gained popularity as compared with the use of abrasives alone or the use of abrasive-chloride mixtures. The primary reason for this popularity, it is believed, is the greater speed with which the straight chemicals provide a cleared wheelpath and roadway. Furthermore, there are indications that the total cost of using chemicals alone, particularly in bulk, may be lower than the cost of using the abrasive-chemical mixtures, which involve handling of much larger quantities of materials.

Few reliable published data are presently available concerning the effectiveness of these materials for actual highway use. Consequently, no recommended application rates based on field tests are available. Application thus far has largely been on a trial and error basis according to the judgment of the individual supervisor in charge of winter maintenance.

This study consisted of two basic parts: ice removal and outdoor storage of chloride salts. The ice removal portion was undertaken to determine the comparative effectiveness of chloride salts and abrasive-chloride salt mixtures in the removal of a controlled amount of ice from pavements under field conditions. The tests were conducted within three temperature ranges and using a controlled amount of vehicular traffic. The outdoor storage portion of the study was undertaken to determine the storage characteristics of sodium chloride, calcium chloride, and mixtures of these chemicals when stored outdoors under polyethylene sheeting. The bulk storage of these materials was of prime concern; however, packaged sodium chloride was included to study the problem of caking in some types of paper bags.

ICE REMOVAL

Test Site

The test site selected for this portion of the study was a short section of unopened Interstate highway. The facility is a four-lane divided portland cement concrete-paved roadway with bituminous-surfaced shoulders. Only one roadway of the divided highway was used for the study, the layout of which is shown in Figure 1.

The test sections were 200 ft in length with a distance of 100 ft between test sections to provide a "track-off" area and to minimize carry-over from one test section to another. An over-all view of the site during a test is shown in Figure 2.

Originally it was planned to run the tests primarily on a bituminous-surfaced roadway; however, difficulties in obtaining a suitable test site prevented this. Only one test was performed on a bituminous pavement, that being in the 1961-62 series of tests on the 10-ft bituminous-surfaced shoulder of the Interstate highway.

Site Preparation

Before each test, the site was cleared of snow using snow plows and power brooms. Next, copper constantan thermocouples, AS & W Wire Gauge No. 25, were installed on the roadway surface to measure the ice temperatures throughout the test periods. Figure 3 shows a typical thermocouple installation with the thermocouple tip about 5 in. from the expansion joint near the black tape.

Two additional thermocouples were used to measure air temperature—one was placed in a 2-in. diameter black tube 12 in. in length; the second was placed in a Florence flask. All thermocouple readings were measured with a Leads and Northrup laboratory potentiometer. For comparison, a mercury thermometer was also used for measuring air temperature. A tabulation of the average ice and air temperatures and cloud cover conditions is given in Table 1.

After installation of the roadway thermocouples, water was applied to the test sections using a trailer-mounted water tank and spray bar distributor. The output of the distributor was calibrated and a reasonable speed of the vehicle determined to apply the necessary quantity of water in several passes to produce an ice thickness approximately $\frac{1}{16}$ in. over a width of about 10 ft. Figure 4 shows the equipment and method used to ice the road. The air temperature at the time of ice formation was generally below 15 F.

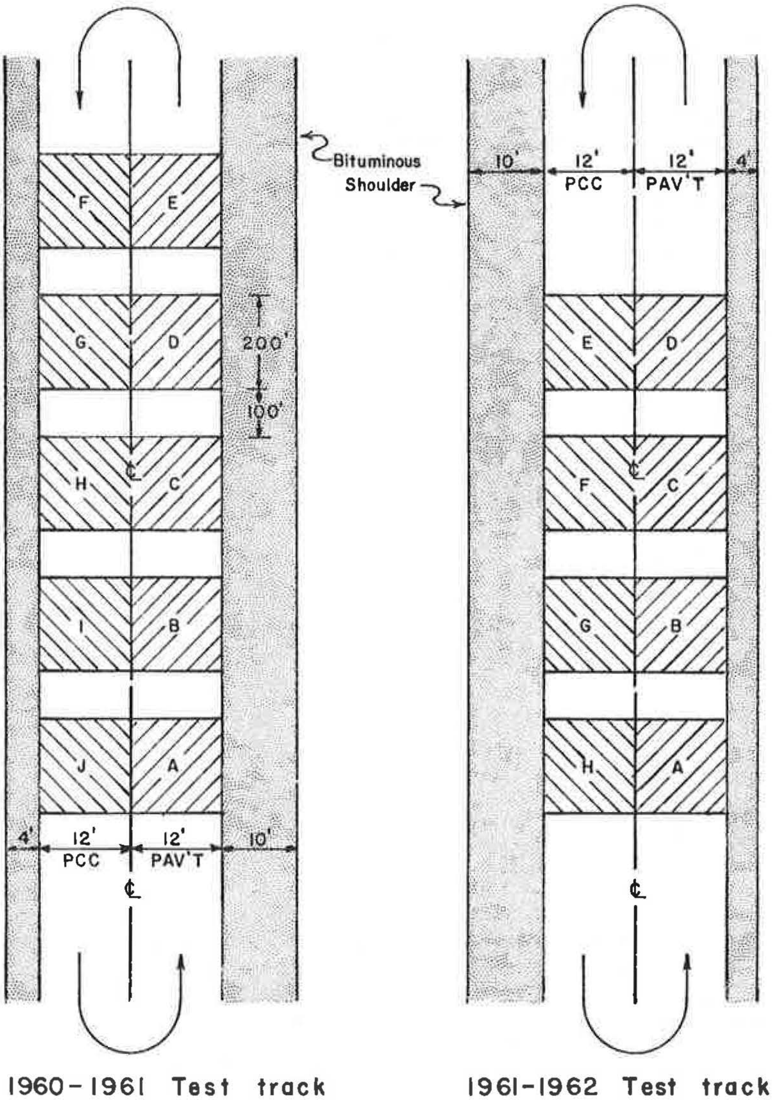


Figure 1. Layout of test sections.

Figure 2. Over-all view of test site.

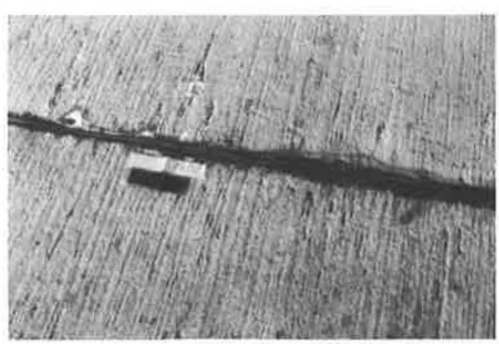


Figure 3. Typical thermocouple installation.

TABLE 1

ICE AND AIR TEMPERATURES

Test No.[a]	Date of Test	Pavement Surface Type[b]	Condition			
			Temperature (^{o}F)			Weather
			Ice	Air	Avg.	
H-1	1-6-61	PCC	32	32	32	Sunny
H-2	1-23-62	Bit.	28	24	26	Variable cloudiness
M-1	1-19-61	PCC	27	12	19.5	Cloudy
M-2	11-11-62	PCC	13	11	12	Cloudy[c]
M-3	2-1-62	PCC	23	12	17.5	Cloudy
M-4	2-7-62	PCC	24	9	16.5	Bright sun
L-1	1-27-61	PCC	15	8	11.5	Partly cloudy
L-2	1-16-62	PCC	12	6	9	Moderate sun
L-3	3-2-62	PCC	15	10	12.5	Slightly sunny

[a]Letter refers to temperature range; number refers to test series.

[b]PCC = portland cement concrete; Bit. = bituminous.

[c]Started snowing during test.

A period of at least 1 hr was provided for the ice to become completely formed before the chemicals and abrasives were applied.

Ice Removal Materials

The ice removal materials selected for comparison in this study were sodium chloride (rock salt and evaporated salt); Type I and Type II calcium chloride flakes; calcium chloride pellets; mixtures of calcium chloride and sodium chloride; a mixture of calcium chloride and sand; and a mixture of sodium chloride and sand. The materials used are identified in Table 2.

The sodium chloride and calcium chloride used in this study met the requirements of AASHO Designation M143-54 and M144-55, respectively. The sand used met the gradation requirements of AASHO Designation M6-51.

Originally, tests were contemplated on a more extensive variety of chemical mixtures. However, after performing three field tests during the winter of 1960-61 and analyzing the results of these; it was decided to eliminate some of the variations in mixtures because the comparative ice removal action between mixtures was not too apparent. At this time it was also decided to include mixtures of chemicals and abrasives in the 1961-62 program of tests because the use of mixtures of these materials is quite common.

Application of Chemicals and Abrasives

In this study the application of chemicals and abrasives was, to a certain extent, a variation of the practices commonly used by maintenance engineers. In the State of Minnesota, straight chemicals are commonly applied in a narrow band along the centerline of two-lane roadways. Abrasives are applied by vehicles straddling the centerline and spreading the material with a disc spinner over both lanes in a single operation; although in some cases, single-lane spreading operations are required and used.

TABLE 2

MATERIALS

Material No.	Sodium Chloride		Calcium Chloride			
	Rock Salt	Evaporated	Pellets[a]	Flakes Type I[b]	Flakes Type II[c]	Sand
1	1					
2			1			
3				1		
4		1				
5	3/4		1/4			
6	2/3		1/3			
7	3/4			1/4		
8	2/3			1/3		
9			1/2			1/2
10	1/2					1/2
11	1/2		1/2			
12	1/2			1/2		
13					1	
14	1/2				1/2	
15	3/4				1/4	

[a]AASHO M 144-57 concentrated pellet calcium chloride, 94 percent minimum purity.
[b]AASHO M 144-57 regular flake calcium chloride, 77 percent minimum purity.
[c]AASHO M 144-57 concentrated flake calcium chloride, 94 percent minimum purity.

Figure 4. Applying water to ice road.

In this study, chemicals were applied in a narrow band approximately 3 ft out from the centerline using the chemical dribbler shown in Figures 5, 6, and 7.

In Figure 6, the application of evaporated salt is shown to produce a very narrow band of chemical on the roadway, whereas in Figure 7 the mixture of calcium chloride and sodium chloride is shown to scatter considerably. Similar scattering was noted with straight calcium chloride pellets as well as with rock salt. The calcium chloride flakes, however,

Figure 5. Chemical dribbler.

Figure 6. Dribbler applying evaporated salt.

Figure 7. Dribbler applying calcium chloride and sodium chloride mixture.

Figure 8. Disc spinner applying abrasive mixture.

scattered less than the calcium chloride pellets or rock salt but more than the evaporated salt.

Equipment used for applying the abrasive mixtures is shown in Figure 8. This equipment produced the least degree of uniformity of applied materials to the roadway although it is probably one of the more extensively used pieces of winter maintenance equipment.

The width of application of abrasives varied from about 3 ft to 10 ft; however, most applications were between 3 and 8 ft wide.

Traffic

Simulated traffic over the test sections consisted of three vehicles of mixed traffic operating with a 1-min headway between vehicles traveling at 15 to 20 mph. The vehicles were one 3-ton dump truck, one ½-ton pickup truck, and one passenger car. This volume of traffic corresponds approximately to 1,440 vehicles per lane per day. Traffic began immediately after all materials were placed and the conditions of spread recorded. Because the vehicle headway remained constant, the data are analyzed on a time basis rather than on vehicle coverage.

Collection of Test Data

The observed data were collected by two rating teams each consisting of two raters. Observations were made and recorded by the rating teams at 15-min intervals throughout the test period. The observations included such items as width, length, and thickness of ice; actual quantity and location of chemical or abrasive applied; formation of brine; and condition of the ice with respect to the amount of removal. A typical rating sheet is shown in Figure 9 and the observed and calculated data are given in Tables 3 and 4.

INF. 604

SECTION E

ICING DATA:

Date: 2/1/62 Air Temp. Beg. 16 °F.
Surf. Condition (Not used) Air Temp. End 15 °F.
Water Applied 600 Ga
Width of Ice 11½' Ft. Thick. of Ice 1/16 In
Remarks

SALT APPLICATION DATA:

Date: 2/1/62 Humidity (Not Determined)
Initial Surf. Cond. (Not Used)

Chemical Used NaCl (Rock Salt) Salt Application Rate 263 #/lane/mile
Rel. Length of Appl/-Beg. +6 Width of Application 3½' to 4½'
End +15 Location of Application 1-5½' & ½'-4'
Traffic: 1440 vplpd Remarks

ICE REMOVAL RATING:

Time	Formation of Brine	Condition of Ice: Total	IWT	OWT	Width Affected	Cleared Area	Flaking Off	Tracking Off Section	General Comments
15	G	—	Hc - Sc	Brine Running	5' 1'-6'	15% IWT	—	80'	—
30	D	—	Sc - Cl	✓	✓	1'-5' 90%	—	100'	—
45	✓	—	Cl	✓	✓	1'-5' 100%	—	✓	S. end 3' cl 1'-4'
1:00	✓	—	✓	Some Slush	✓	✓	—	✓	✓
1:15	✓	—	✓	✓	✓	✓	—	✓	✓
1:30									
1:45									
2:00									
2:15									
2:30									
2:45									
3:00									
3:15									
3:30									
3:45									
4:00									

Note:
G = Good
D = Done
Hc = Honeycombed
Sc = Scabby
Cl = Clear

Figure 9. Typical rating sheet.

TABLE 3

OBSERVED AND CALCULATED DATA

Icing Data				Chemical and Abrasive Application						
Test No.[a]	Average Width (ft)	Computed Thickness (in.)	Planned (lb per lane-mile)	Test Section Applied		Measured Width (ft)	Measured Length (ft)		Computed Weight[c]	
				60-61[b]	61-62		Total	On Ice	Lb per Lane-Mile	Lb per Sq Yd
H-1-1	9	0.075	250	A		4.0	208	187	240	0.10
H-2-1	8	0.056	275		E	3.5	157	157	350	0.17
M-1-1	10	0.075	250	A		4.0	190	190	263	0.11
M-2-1	9	0.077	275		E	4.5	215	200	256	0.10
M-3-1	12	0.045	275		E	4.0	209	194	263	0.11
M-4-1	9	0.078	275		E	4.5	137	137	402	0.15
L-1-1	9	0.075	250	A		4.0	170	170	294	0.13
L-2-1	8	0.055	350		E	4.0	139	139	503	0.21
L-3-1	8	0.065	350		E	2.5	242	197	289	0.19
H-1-2	9	0.075	250	D		4.0	235	194	213	0.09
H-2-2	8	0.056	275		A	3.5	198	198	278	0.14
M-1-2	10	0.075	250	D		2.5	223	190	224	0.15
M-2-2	9	0.077	275		A	3.5	204	200	270	0.13
M-3-2	12	0.045	275		A	5.0	202	197	272	0.09
M-4-2	9	0.078	275		A	4.5	175	175	314	0.12
L-1-2	10	0.075	250	D		4.0	240	200	208	0.09
L-2-2	8	0.055	350		A	3.5	235	200	298	0.15
L-3-2	9.5	0.065	350		A	6.0	200	200	350	0.10
H-1-3	9	0.075	250	J		3.0	231	200	217	0.12
H-2-3	8	0.056	275		B	3.0	147	141	374	0.21
M-1-3	10	0.075	250	J		3.0	186	186	269	0.15
M-2-3	9	0.077	275		B	3.5	212	200	259	0.13
M-3-3	12	0.035	275		B	4.0	187	187	294	0.13
M-4-3	9	0.078	275		B	3.0	160	160	344	0.19
L-1-3		0.075	250	J		1.5	202	198	248	0.29
L-2-3	8	0.055	350		B	2.8	162	162	432	0.27
L-3-3	9.5	0.065	350		B	3.5	180	180	389	0.19
H-2-4	8	0.056	275		G	3.0	169	169	325	0.19
M-2-4	9	0.077	275		G	4.0	170	170	324	0.14
M-3-4	12	0.045	275		G	2.5	202	196	136	0.19
M-4-4	9	0.078	275		G	2.5	202	200	272	0.19
L-2-4	8	0.055	350		G	2.5	144	144	486	0.33
L-3-4	9.5	0.065	350		G	2.0	220	200	318	0.27
H-1-5	9	0.075	250	B		6.0	227	180	250	0.06
H-2-6	8	0.056	275		F	3.5	137	137	401	0.20
M-1-5	10	0.075	250	B		4.0	200	200	250	0.16
M-2-6	9	0.077	275		F	4.0	182	182	302	0.16
M-3-6	12	0.045	275		F	4.0	174	174	302	0.13
M-4-6	9	0.078	275		F	6.0	150	150	367	0.14
L-1-5	9	0.075	250	B		4.5	191	191	263	0.10
L-2-6	8	0.055	350		F	4.3	143	143	490	0.19
L-3-6	9.5	0.065	350		F	3.5	210	200	333	0.16
H-1-7	9	0.075	250	G		2.5	226	200	250	0.17
H-2-8	8	0.056	275		C	3.5	139	139	396	0.19
M-1-7	10	0.075	250	G		3.5	189	189	265	0.13
M-2-8	10	0.077	275		C	4.0	196	196	281	0.12
M-3-8	11	0.045	275		C	4.0	181	181	304	0.13
M-4-8	9	0.078	275		C	2.5	137	137	401	0.27
L-1-7	9	0.075	250	G		3.0	191	185	279	0.15
L-2-8	8	0.055	350		C	2.5	140	140	500	0.34
L-3-8	9.5	0.065	350		C	5.0	204	200	344	0.34
H-2-9	8	0.056	275		H	10.0	241	200	157	0.04
M-2-9	9	0.077	275		H	7.0	210	190	261	0.08
M-3-9	11.5	0.045	275		H	9.5	227	200	167	0.04
M-4-9	9.5	0.078	275		H	5.0	215	190	177	0.09
L-2-9	8	0.055	350		H	2.8	195	177	244	0.15
L-3-9	9.5	0.065	350		H	5.5	191	191	249	0.08
H-2-10	8	0.056	275		D	8.0	133	133	323	0.07
M-2-10	10	0.077	275		D	5.0	189	159	228	0.08
M-3-10	10.5	0.045	275		D	3.0	221	200	194	0.11
M-4-10	9	0.078	275		D	5.5	215	200	200	0.06
L-2-10	8	0.055	350		D	2.7	183	183	376	0.24
L-3-10	9.5	0.065	350		D	4.0	210	170	256	0.11
L-1-11	9	0.075	250	C		4.0	207	200	258	0.11
H-1-11	9	0.075	250	C		7.0	231	179	259	0.06
M-1-11	10	0.075	250	C		4.0	210	193	230	0.10
L-1-12	9	0.075	250	I		3.0	200	200	250	0.14
H-1-12	9	0.075	250	I		2.5	255	200	216	0.14
M-1-12	10	0.075	250	I		4.0	206	194	243	0.10
L-1-13	9	0.075	250	E		1.5	193	193	258	0.29
H-1-13	9	0.075	250	E		1.5	224	194	223	0.25
M-1-13	10	0.075	250	E		1.5	206	200	243	0.27
L-1-14	9	0.075	250	H		3.5	200	200	250	0.12
H-1-14	9	0.075	250	H		2.5	231	188	222	0.15
M-1-14	10	0.075	250	H		3.5	153	153	325	0.15
L-1-15	9	0.075	250	F		3.0	208	193	258	0.15
H-1-15	9	0.075	250	F		2.0	246	200	203	0.17
M-1-15	10	0.075	250	F		4.0	196	196	255	0.11

[a]Letter refers to temperature range; first number refers to test series; second number refers to material type (Table 2).

[b]Years 1960 and 1961.

[c]Actually applied to the ice.

TABLE 4

OBSERVED ICE REMOVAL DATA

Test No.[a]	100% Brine Formation (min)	Time Required for Ice Removal in 18-In. Wheelpath (min)					Time (min) for Outer Wheel Track to Have	
		10% (Pitted)	15% (Pocked)	40% (Honeycombed)	55% (Scabby)	80% (Clear)	55% Removal (Scabby)	80% Removal (Clear)
H-1-1	45	15	20		30	30	45	90
H-2-1	30				15	30	75	-[b]
M-1-1	30		10	45	75	-[b]		
M-2-1	-[c]	15	30	-[b]				
M-3-1	30			10	30	30	-[b]	
M-4-1	45		15	45	-[b]			
L-1-1	-[c]	15	50	-[b]				
L-2-1	60		15	40	-[b]			
L-3-1	45			15	45	-[b]		
H-1-2	45				15	30	45	75
H-2-2	30				15	30	-[b]	
M-1-2	30			15	-[b]			
M-2-2	-[c]		15	40	60	-[b]		
M-3-2	30				15	15	-[b]	
M-4-2	45			15	45	-[b]		
L-1-2	-[c]	10	25	45	-[b]			
L-2-2	60			10	30	45		
L-3-2	60		15	30	45	90		
H-1-3	45	15	20	30	30	45	45	90
H-2-3	30				15	30	-[b]	
M-1-3	45			15	-[b]			
M-2-3	30		10	-[b]				
M-3-3	20				15	15	-[b]	
M-4-3	30			15	45	-[b]		
L-1-3	-[c]	15	25	50	-[b]			
L-2-3	60			10	15	45		
L-3-3	30		10	25	60	-[b]		
H-2-4	30				15	15	-[b]	
M-2-4	-[c]	-[b]						
M-3-4	30				15	45		
M-4-4	15				15	60		
L-2-4	30				15	75		
L-3-4	45				15	60		
H-1-5	45				15	30	75	120
H-2-6	30				15	15	-[b]	
M-1-5	45		10	45	120	-[b]		
M-2-6	30					15	-[b]	
M-3-6	30			10	15	30	-[b]	
M-4-6	-[c]		15	40	75	-[b]		
L-1-5	-[c]	15	40	-[b]				
L-2-6	75		15	30	45	-[b]		
L-3-6	45				15	30		
H-1-7	30	15	20	30	45	60	60	75
H-2-8	20				20	30	-[b]	
M-1-7	60		15	50	-[b]			
M-2-8	-[c]	10	20	40	-[b]		-[b]	
M-3-8	30					15		
M-4-8	45			15	30	30	-[b]	
L-1-7	-[c]	15	25	75	90	-[b]		
L-2-8	75			15	30	-[b]		
L-3-8	60		15	45	-[b]			
H-2-9	-[c]		15	-[b]				
M-2-9	-[c]	15	30	-[b]				
M-3-9	45			15	45	-[b]		
M-4-9	-[c]	15	25	75	-[b]			
L-2-9	-[c]	15	30	45	-[b]			
L-3-9	60		30	45	60	90		
H-2-10	30				15	45		
M-2-10	-[c]	15	-[b]					
M-3-10	30				15	45		
M-4-10	-[c]		15	40	-[b]			
L-2-10	-[c]	5	15	60	-[b]			
L-3-10	60	15	60	[b]				
L-1-11	60	15	30	-[b]				
H-1-11	15	30	-[b]					
M-1-11	45	15	30	60	-[b]			
L-1-12	-[c]	15	45	-[b]				
H-1-12	45	15	-[b]					
M-1-12	30	15		30	-[b]			
L-1-13	45	15	-[b]					
H-1-13	45					30		
M-1-13	30			15	60	-[b]		
L-1-14	45		15	30	45	-[b]		
H-1-14	-[c]	10	30	45	-[b]			
M-1-14	60	15		30	60	-[b]		
L-1-15	-[c]	10	30	-[b]				
H-1-15	60	15	-[b]					
M-1-15	60	15	30	75	-[b]			

[a]Letter refers to temperature range; first number refers to test series; second number refers to material type (Table 2).

[b]Condition not achieved within time period of test.

[c]Formation of brine incomplete at end of test.

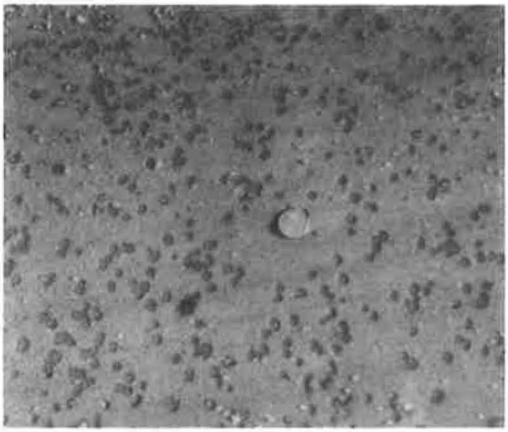


Figure 10. Pocked condition.

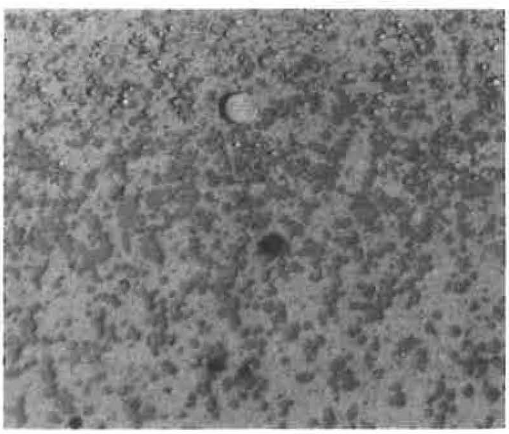


Figure 11. Honeycombed condition.

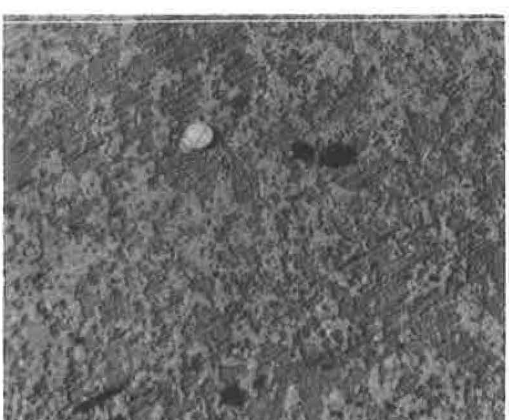


Figure 12. Scabby condition.

Figure 9 and Table 4 show that certain arbitrary standards were established to record the degree of ice removal. The first of these standards was called "pitted." This condition existed when only the ice immediately under the particles of chemical became melted. The second condition was called "pocked" (Fig. 10). In this case, the initial pits have enlarged and bare pavement is beginning to show through.

As melting continues, the holes begin to interconnect as shown in Figure 11 and the third condition, "honeycombed", is achieved. Approximately 40 percent of the ice has melted when the condition "honeycombed" exists.

The fourth condition was called "scabby" (Fig. 12). In this condition only separate fragments of the ice remain on the roadway and approximately 55 percent of the ice has melted. Beyond the "scabby" condition, a "clear" condition was usually achieved where at least 80 percent of the ice had melted and a definite, clear wheelpath was provided.

Temperature Ranges

For the purpose of this report only three temperature ranges are considered; in each case, the temperature of the air is the controlling factor. The low temperature range (0 to 10 F), medium (10 to 20 F), and high (20 to 32 F) are referred to as L, M, and H, respectively. As an example, L-3 would indicate the third test series in the lowest temperature range. It was observed that the ice temperature as measured by the thermocouple was generally significantly higher than the air temperature.

Analysis of Data

Because this portion of the study had so many uncontrolled variables (such as temperature, wind, and humidity), the results are not repeatable. Therefore, the graph plots presented are considered trends based on mean values of the observed and calculated data.

Figure 13 shows a plot of the volume of ice melted by sodium chloride (rock salt) in cubic feet per lane-mile vs time in minutes (vehicle headway remaining constant) for three conditions of average ice and air temperature.

As indicated by the curves, sodium

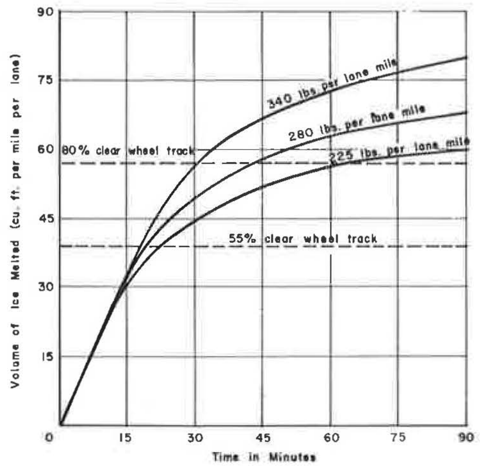


Figure 13. Rate of ice removal for NaCl (rock salt).

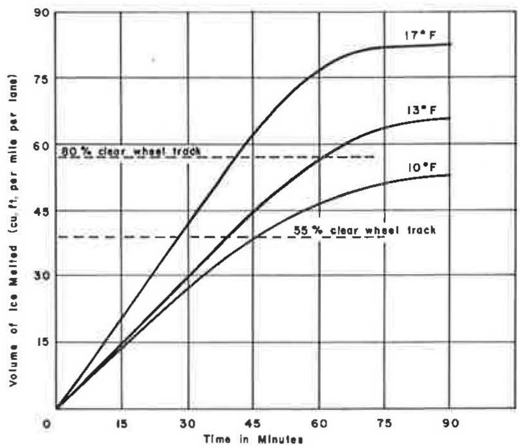


Figure 14. Rate of ice removal by calcium chloride with temperature adjustment.

chloride, with its relatively high eutectic point of -6 F, appears to melt ice at a rate highly dependent on the average of the ice and air temperature. From the observed data, an equation was developed to give a reasonable estimate of the volume of ice that can be melted at a terminal period of 90 min and within a temperature range of 10 to 20 F:

$$V = \frac{R^{1/2}\, T^{1.5}\, k}{I} \tag{1}$$

in which

V = volume of ice melted (cubic feet per lane-mile);
R = rate of applied salt (pounds per lane-mile);
T = average of air and ice temperature ($^{\circ}$F);
I = thickness of ice (inches); and
k = constant with a value of 32×10^{-4}.

The small exponent of R indicates that the rate of rock salt application is of small concern in the rate of ice removal. On the other hand, the large exponent for T indicates that in the case of rock salt, the amount of melt is primarily dependent on the average of ice and air temperatures. The equation also shows that as the ice thickness increases the total amount of melt decreases.

Calcium chloride pellets and flakes have an eutectic point of -58.5 F. Therefore, they do not appear to be as dependent on temperature as sodium chloride for their ice removal rate. Figure 14 is a plot of the volumes of ice melted by calcium chloride pellets at three application rates and with the curves adjusted for temperature differences. The curves show that the amount of thawing is primarily dependent on the amount of calcium chloride applied.

An end-point equation showing this relationship for volume of ice melted with $CaCl_2$ was derived:

$$V = \frac{R^2\, T\, k}{I^{0.8}} \tag{2}$$

in which

V = volume of ice melted (cublic feet per lane-mile);

R = rate of application of chloride (pounds per lane-mile);
T = average of air and ice temperature (^{0}F);
I = thickness of ice (inches);
k = constant with a value of 6.15×10^{-6}.

In Eq. 2 the rate of ice removal varies as the square of the rate of application of calcium chloride. Therefore, it appears that the more chemical applied, the faster the ice will melt.

The relative rates of ice removal by calcium chloride and sodium chloride are shown in Figures 15, 16, and 17. The comparison at 17 F shows that initially the $CaCl_2$ is more effective but as the action progresses the rate of melting decreases whereas that of the rock salt remains more nearly constant up to about 1 hr and eventually overtakes the rate of the calcium chloride. As shown in Figure 16, the rate of ice removal by rock salt at lower temperatures does not approach that of the $CaCl_2$ pellets. From Figures 16 and 17 it may be concluded that sodium chloride is relatively ineffective below 10 F in clearing a wheelpath within a reasonable period of time.

Figures 18 and 19 are plots of the comparative ice removal rates of calcium chloride (pellets), sodium chloride (rock salt), and a ⅓ calcium chloride (pellet) to ⅔ sodium chloride (rock salt) mixture at 17 and 10 F, respectively. The plots show that the chemical mixture removes ice at a rate closely resembling that of straight calcium chloride while still retaining most of the economical advantage of sodium chloride.

The comparative ice removal rates of sodium chloride (evaporated salt) and the ⅓ $CaCl_2$ to ⅔ NaCl mixture at 17 and 10 F are shown in Figures 20 and 21, respectively. The plots show that initially the rate of ice removal by the evaporated salt is somewhat greater than the pellet-rock salt mixture; however, the evaporated salt is overtaken by the mixture within 30 to 40 min.

Figures 22 and 23 are plots of the comparative ice removal rates of abrasive-chloride mixtures and a ⅓ $CaCl_2$ to ⅔ NaCl mixture. As indicated by the curves the abrasive-chloride mixtures are relatively ineffective for ice removal from pavements particularly at the lower temperatures.

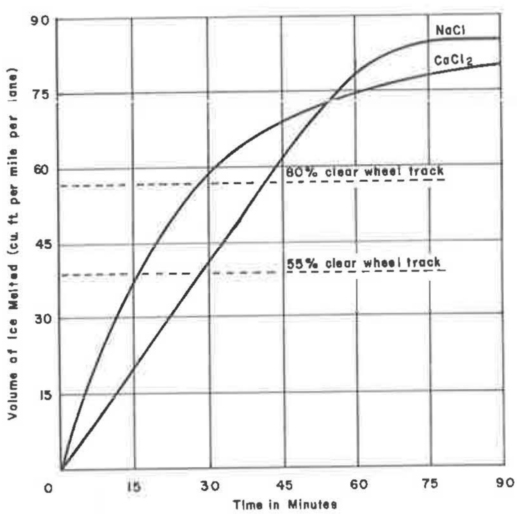


Figure 15. Rate of ice removal by $CaCl_2$ pellets and rock salt at 17 F.

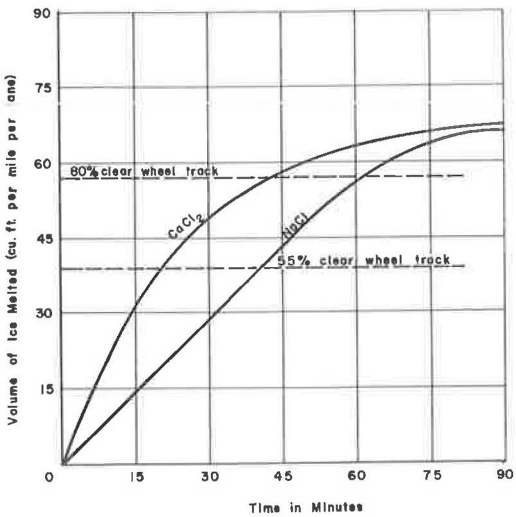


Figure 16. Rate of ice removal by $CaCl_2$ pellets and rock salt at 13 F.

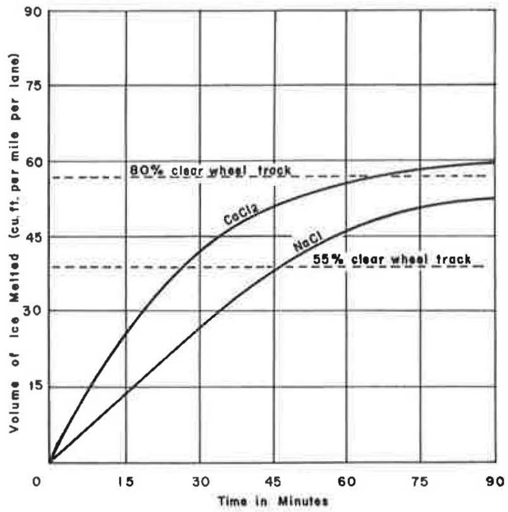


Figure 17. Rate of ice removal by $CaCl_2$ pellets and rock salt at 10 F.

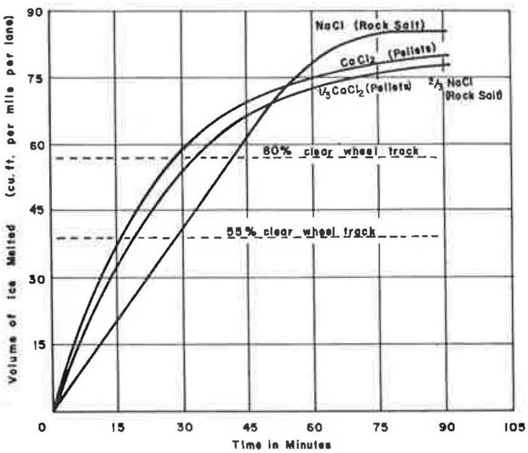


Figure 18. Rate of ice removal by $CaCl_2$ (pellets), NaCl (rock salt), and $\frac{1}{3}$ $CaCl_2$ to $\frac{2}{3}$ NaCl at 17 F.

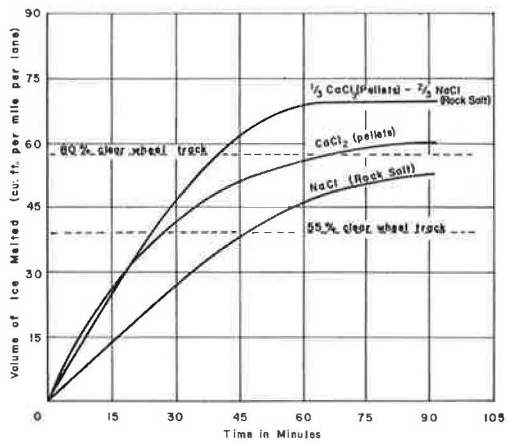


Figure 19. Rate of ice removal by $CaCl_2$ (pellets), NaCl (rock salt), and $\frac{1}{3}$ $CaCl_2$ to $\frac{2}{3}$ NaCl at 10 F.

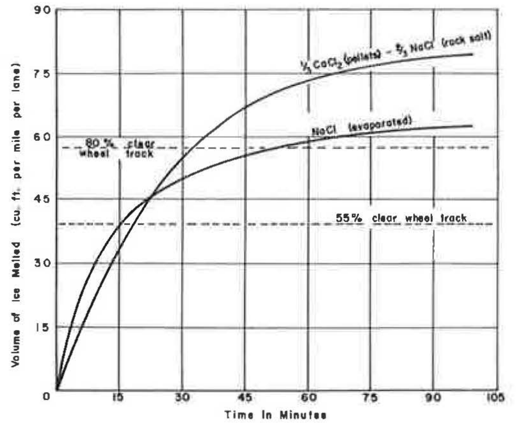


Figure 20. Rate of ice removal by NaCl (evaporated) and $\frac{1}{3}$ $CaCl_2$ (pellets) to $\frac{2}{3}$ NaCl (rock salt) at 17 F.

Ice Removal Conclusions

On the basis of the limited test data for this study, the following general conclusions are indicated:

1. The amount of melt by sodium chloride (rock salt) is primarily dependent on the average of the ice and air temperatures.
2. The amount of melt by calcium chloride within a given temperature range is proportionate to the amount of chemical applied.
3. The rate of ice removal for both calcium chloride and sodium chloride is similar above 15 F.
4. Below 15 F, calcium chloride was found to be more effective than sodium chloride.

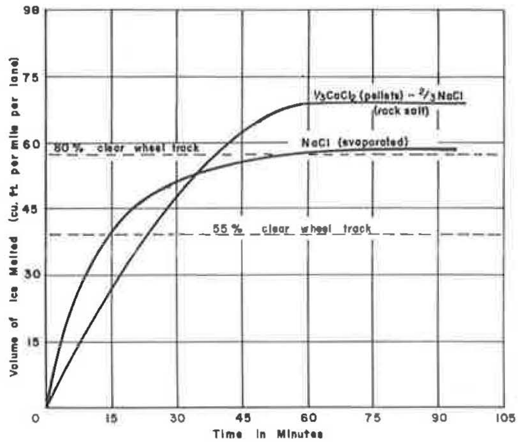


Figure 21. Rate of ice removal by NaCl (evaporated) and $\frac{1}{3}$ $CaCl_2$ (pellets) to $\frac{2}{3}$ NaCl (rock salt) at 10 F.

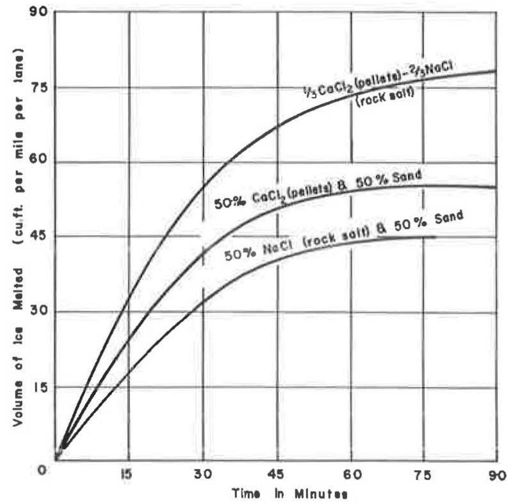


Figure 22. Rate of ice removal by abrasive-chloride mixtures and $CaCl_2$-NaCl mixture at 17 F.

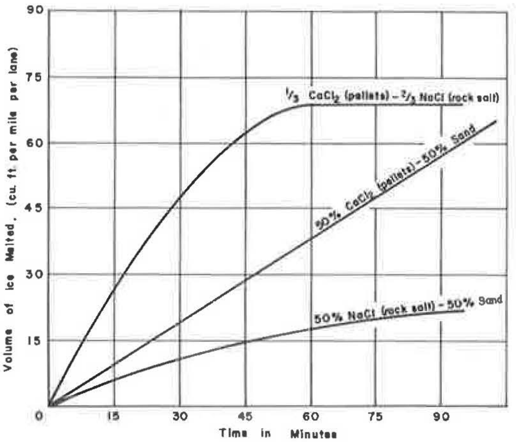


Figure 23. Rate of ice removal by abrasive-chloride mixtures and $CaCl_2$-NaCl mixture at 10 F.

5. Below 10 F, sodium chloride was found to be relatively ineffective in clearing a wheelpath within a reasonable time period of 60 min.

6. The rate of application of sodium chloride should be varied inversely with the temperature.

7. The rate at which both calcium chloride and sodium chloride removes ice is inversely proportional to the ice thickness.

8. Calcium chloride pellets were found to remove ice more rapidly than either calcium chloride flakes or rock salt.

9. A $\frac{1}{3}$ $CaCl_2$ to $\frac{2}{3}$ NaCl mixture was found to be the only one that provided clear wheelpaths with any consistency. The mixture removed ice at a rate closely resembling that of straight calcium chloride.

10. Sodium chloride (evaporated salt) was found to have a high rate of ice removal in the first 20 min after application, but fell off rapidly after that time.

11. Mixtures of sodium chloride with abrasive and calcium chloride with abrasive were found to be relatively ineffective for ice removal from pavements particularly at temperatures below about 15 F.

OUTDOOR STORAGE OF CHLORIDE SALTS

The primary concern of this portion of the study was the determination of the storage characteristics of bulk sodium chloride, calcium chloride, and mixtures of the two. The decision to include packaged material in the study was based on the problem of salt caking in some types of paper bags. Packaged calcium chloride was not included in the study because no problem was evident in this respect.

Test Site

The test site selected for this portion of the study was within the storage grounds of a highway maintenance depot. The tests were conducted on a bituminous-surfaced storage base 10 ft wide and 60 ft long. The base had a slope in only one direction of ½ in. per ft across the 10-ft width. The site and materials under test were protected from accidential disturbance by a 4-ft high wood slat snow fence. An over-all view of the test site is shown in Figure 24.

Figure 24. Test site for outdoor storage of chloride salt.

Test Materials

The bulk materials included in the tests are given in Table 5. It was assumed the sodium chloride (rock salt) had a purity of 100 percent and the weights of calcium chloride were computed based on their purity as advertised by the manufacturer.

Where mixtures were included they were prepared by hand using square-nosed shovels to blend the materials.

The packaged material consisted of 40 bags of sodium chloride (rock salt) in several types of paper bags:

Figure 25. General arrangement of packaged NaCl.

1. U—untreated paper bags with three layers of untreated paper (15 bags).
2. B—bituminous-treated bags with three layers of paper, two of which were treated with bituminous (10 bags).
3. Bo—bituminous-treated bags similar to B except they were in previous storage for about nine months (5 bags).
4. P—plastic cemented bags having three layers of paper, two of which were glued together with a plastic cement (10 bags).

The packaged material was placed on planks in five stacks of eight bags per stack. Because there were five stacks, an arrangement was worked out so that no more than two similar bags would be represented in each layer. The arrangement of stacking is shown in Figure 25.

All materials included in the study were placed in test between November 7 and 10, 1960, except test material G (100 percent sodium chloride with a sand cover) which was placed May 3, 1961.

Protective Covering Material

The protective covering material used in this study was a nominal 5-mil white polyethylene sheeting similar to that used for curing concrete. It was anchored with sand-filled burlap bags placed at the base of each storage pile. No polyethylene sheeting material was used to cover test material G because this test was introduced to study the effectiveness of a light sand cover.

TABLE 5

POUNDS OF BULK MATERIALS USED

Identification	Sodium-Chloride Rock Salt (lb)	Calcium Chloride (lb)	
		Type I Flakes	Pellets
A	1,000		
B		1,282	
C			1,053
D	750		263
E	500		526
F	500	641	
G	1,000[a]		

[a]Covered with a 4-in. layer of sand instead of polyethylene sheeting.

Collection of Data

Observations of the bulk test materials were made once a month and those for the packaged material once every two months. Observations included moisture determinations, amounts of caking and crusting, and determination on whether each material was free flowing.

Duplicate moisture samples of the bulk materials were taken about midway between the base and apex of each conical storage pile. They were taken either at a depth of 2 to 3 in. below the surface or one sample was taken at the surface and the other about 5 to 6 in. into the pile. The moisture determinations for the bulk materials are given in Table 6; the figures given are averages of the separate tests.

Single moisture samples were taken of the bagged material by cutting a small opening near the center of the bag, as shown in Figure 26, removing about 300 g of salt and then taping the opening closed.

Each bag of sodium chloride was sampled at least twice during the test period and almost one-half were sampled three times or more. The moisture data and date of moisture test are given in Table 7.

Some slight caking was found to exist in only a few of the bags of rock salt within the 10-month storage period. However, the cake was easily broken by hand and no difficulty would have been experienced in using the material.

On the other hand, considerable caking and crusting did occur in some of the bulk storage piles which would create difficulty in their use. The observations of crusting and caking are summarized in Table 8 along with the average moisture content of each material.

Analysis of Data

As shown by the data, 100 percent sodium-chloride rock salt (test material A) crusted within a two-month storage period; it was completely caked within ten months although the moisture content remained quite low (0.6 to 0.9 percent).

The 100 percent calcium chloride, Type I flakes, (test material B) developed a base crust of 1 to 1½ in. within a two-month storage period while no surface crusting was evident even though the moisture content was quite high (11.9 percent). At ten months, a slight outer surface crust developed which was easily broken. The chemical remained free flowing in spite of the fact that the moisture content almost doubled in an eight-month period. It is believed the base crust formation in this and other stockpiles in the test, as shown in Figures 27 and 28, is related to the amount of surface water on the storage base that is available to the stockpile. This points up the need for a well-constructed, free-draining storage base.

TABLE 6

MOISTURE DATA FOR BULK MATERIALS

Test Material	Average Moisture (percent of dry weight)											
	12-6-60	1-3-61	2-3-61	3-1-61	4-7-61	5-3-61	6-1-61	7-6-61	8-7-61	9-6-61	10-4-61	12-7-61
A	0.8	0.5	0.7	0.9	0.6	0.2	0.9	2.2	1.1	0.9	2.5	-
B	11.7	11.9	11.1	11.8	10.3	11.9	15.6	16.0	16.9	22.8	23.6	-
C	0.6	0.4	0.4	0.6	0.1	0.4	0.8	0.8	0.6	1.3	1.7	-
D	0.6	0.6	0.5	1.1	0.6	0.4	0.6	1.1	1.3	1.5	5.8	-
E	0.4	0.5	0.3	0.5	0.3	0.4	0.4	0.9	0.6	1.7	6.4	-
F	6.7	8.0	6.7	7.2	6.2	8.3	10.7	14.2	12.8	14.5	21.5	-
G	-	-	-	-	-	0.2	0.5	2.6	0.6	0.5	1.0	1.0

Figure 26. Method of taking moisture sample from packaged NaCl.

Figure 27. Base crust formation in rock salt.

Figure 28. Base crust formation in calcium chloride.

The 100 percent calcium chloride pellets (test material C) developed a base crust of ½ to 1½ in. within a two-month storage period but no surface crusting occurred. At ten months, no increase in base crust was noted although a crust varying from 0 to ½ in. developed on the surface. The moisture content remained low (1.3 percent) and the chemical was free flowing.

Mixtures of sodium chloride and calcium chloride all developed a base crust of 0 to 1½ in. within a two-month storage period with no surface crusting at all. At ten months the base crust varied from 1 to 2 in. with only slight surface crusting. All chemical mixtures remained in a free flowing condition condition throughout the test period. The moisture contents remained below 2 percent except in test material F (50 percent sodium chloride and 50 percent calcium chloride flakes) which had a gradual increase to 15 percent at the end of ten months. In all cases of the mixtures, after a short period of storage only particles of rock salt were left on the surface. These particles then fused together to form this light crust.

Sodium chloride with a 4-in. sand cover, (test material G) developed into a fairly hard cake with a two-month period and at the end of seven months became very hard on the surface. The material was not free flowing and this method of storage cannot be considered desirable.

Packaged rock salt stored well. The moisture content generally remained under 1 percent during the ten-month storage period with no detrimental amount of hardening or caking occurring. Only in a few cases where the moisture content approached or exceeded 1 percent was any caking noticed and in these cases the cake was easily

TABLE 7

MOISTURE DATA FOR PACKAGED NaCl

Bag Position[a]	Moisture (percent of dry weight)					
	2-3-61	4-7-61	6-2-61	8-7-61	10-4-61	11-8-61
1 NB		0.2				0.4
1 EB			0.3			0.3
1 SU			0.9			0.4
1 WU		0.4				0.5
1 CP		0.1				0.4
2 NP			0.4			0.2
2 EP			0.5			0.3
2 SB			0.6			0.2
2 WU			0.4			0.4
2 CU		0.3				0.4
3 NU			0.5			0.6
3 EU			0.5			0.2
3 SP			0.5			0.3
3 WB			0.3			0.2
3 CB		0.3				0.2
4 NBo			0.4			0.1
4 EB			0.3			0.2
4 SU			0.4			0.1
4 WP			0.3			0.3
4 CP		0.1		0.2		0.2
5 NP	0.3			0.1		0.2
5 EBo	0.3			0.3		0.2
5 SB	0.3			0.2		0.2
5 WU	0.4			0.1		0.4
5 CU	0.4			0.4		0.3
6 NU	0.6					0.6
6 EU			0.4			0.4
6 SP		0.2		0.3		0.3
6 WB			0.3			0.3
6 CB	0.3			0.1		0.2
7 NBo		0.1			0.3	0.3
7 EB		0.1			0.1	0.5
7 SU		0.6			0.2	0.4
7 WU			0.3		0.4	0.4
7 CP	0.3			0.1	0.2	0.3
8 NP	0.4			0.3	0.3	0.5
8 EBo	0.3			0.4	0.1	0.2
8 SB	0.4			0.2	0.3	0.2
8 WU	0.4			0.6	0.6	0.8
8 CU		0.4			0.9	1.2

[a]Number indicates layer within stack; first letter refers to compass position of stack (north, east, south, west, or center); second letter refers to type of paper bag as identified in text.

TABLE 8

SUMMARY OF BULK STORAGE PILE CONDITIONS

Test Material	After Two Months Storage: Physical Condition	After Two Months Storage: Moisture (avg %)	After Ten[a] Months Storage: Physical Condition	After Ten[a] Months Storage: Moisture (avg %)
A	2- to 3-in. hard base crust; slight crust on outer surface, easily broken	0.6	Entire pile caked in solid mass; broken only with some difficulty	0.9
B	1- to 1½-in. base crust; no surface crust, free flowing	11.9	Slight outer surface crust, easily broken and free flowing beneath	22.8
C	½- to 1½-in. base crust; no surface crust, free flowing	0.4	1- to 1½-in. base crust; 0- to ½-in. surface crust, free flowing beneath	1.3
D	0- to 1-in. base crust; no outer surface crust, free flowing	0.6	1-in. base crust; slight crusting at outer surface, free flowing beneath	1.5
E	1-in. base crust; no surface crust, free flowing	0.5	1-in. base crust; slight crusting in outer surface, free flowing beneath	1.7
F	1- to 1½-in. base crust; no outer surface crust, free flowing	8.0	2-in. base crust; no crusting at outer surface, free flowing	15.0
G	Fairly hard cake developing especially on outside, softer within	2.6	2- to 3-in. very hard surface crust; inner portion caked but not too hard	1.0

[a]All except test material G which was seven months.

broken by hand. A marked difference in moisture is noted between the types of paper bags. The moisture of the salt in the unlined bags was as much as double that in the treated bags. The polyethylene sheeting used in this study as a cover over the bags provided a very effective moisture barrier. In fact, the barrier was so effective that it prevented the natural escape of moisture during dry periods. The moisture then collected on the outside of the bags as shown in Figure 29 and, in some cases, deteriorated the bags to the point of breaking.

Figure 29. Collection of moisture on bags.

Outdoor Storage Conclusions

1. Calcium chloride (pellets or

flakes), and mixtures of sodium chloride with at least 25 percent calcium chloride, can be stored outdoors in bulk with a light polyethylene sheet cover for at least ten months without appreciable hardening.

2. Sodium chloride (rock salt) can be stored outside up to two months in bulk when covered with a light polyethylene sheeting. Bulk storage of sodium chloride beyond three months appears to be unsatisfactory.

3. Packaged sodium chloride (100-lb bags) may be stored outdoors with a light polyethylene sheet cover for at least ten months with no appreciable hardening.

RECOMMENDATIONS

Ice Removal

Table 9 gives a recommended range of application rate for eight ice removal materials. These rates are based on an ice thickness of about 1/16 in. and on clearing a wheelpath 1½ to 3 ft in width. However, additional material may be required for complete removal of ice from the roadway.

Because the application rates recommended are based on a limited number of tests, they may require modification after being used by the maintenance crews for a suitable period of time. Suggested modifications are therefore invited.

Outdoor Storage of Chloride Salts

Bulk sodium chloride intended for outdoor storage longer than two months should be mixed with at least 25 percent calcium chloride—preferably pellets. A mixture of ⅓ calcium chloride (pellets) to ⅔ sodium chloride (rock salt) appears to be the best all-round mixture when considering storage, ice removal action, and economy.

When bagged chloride salts are stored outdoors under polyethylene sheeting, provisisons should be made for ventilation to eliminate the collection of moisture on the paper bags.

Further Research

Additional testing of the more promising materials to be conducted under field conditions is recommended. It is suggested that the roadway be iced in a manner similar to that used in this study. However, more thought should be given to the

TABLE 9

RECOMMENDED APPLICATION RATES OF CHEMICALS AND CHEMICAL-ABRASIVES FOR ICE REMOVAL

Ice Removal Material	Suggested Width of Spread (ft)	Application Rates (lb per lane-mile[a]) for 1/16-In. Ice		
		Below 10 F	10-20 F	20-32 F
$CaCl_2$ (pellets)	2-4	300-375	250-300	(175-250)[b]
$CaCl_2$ (flakes)	2-4	350-450	275-350	(200-275)[b]
NaCl (rock salt)	3-4	(400-550)[c]	250-400	200-250
NaCl (evaporated)	3	(325-500)[c]	200-325	150-200
⅓ $CaCl_2$ (pellets) - ⅔ NaCl (rock salt)	2-4	300-475	250-300	175-250
⅓ $CaCl_2$ (flakes) - ⅔ NaCl (rock salt)	2-4	350-500	275-350	200-275
50% $CaCl_2$ (pellets) - 50% sand	4	(excess of 500)[c]	(300-500)[c]	200-300
50% NaCl (rock salt) - 50% sand	4	(excess of 600)[c]	(325-600)[c]	225-325

[a]Quantities given doubled for 2-lane roadways.
[b]Not recommended because greasy condition often results; quantities given are suggested if no other material is available.
[c]Not recommended because of low rate of ice removal; quantities given are suggested if no other material is available.

elimination or greater control of some of the variables. For example, the chemicals and chemical mixtures might be applied at a better controlled rate using a fertilizer spreader; also, a better method for evaluating the amount of ice removal might be found.

Other governmental bodies and agencies are encouraged to conduct field tests similar to Minnesota's to validate or disprove the results and conclusions presented herein.

REFERENCE

1. Tiney, B.C., "Treatment of Icy Pavements." HRB Proc., 11:364-366, pt. I, (1931).

Discussion

ALAN K. JEYDEL, Technical Director, Salt Institute, Chicago, Illinois—The Minnesota report is commendable for attempting to simulate actual road ice conditions while providing for measurements of temperature, ice thickness, and rates of application of the various chemicals and mixtures.

Some of the conclusions and recommendations drawn from the data do not seem justified and are at variance with test data. For instance, on the basis of the test data the efficacy of salt at temperatures down to 10 F is at least equal to the mixture. The report cites mixtures for economy, storage, and effectiveness over straight chemicals. It is questionable whether there are data to support these broad claimed advantages.

As for economy, there are no cost figures shown to verify statements made and it is difficult to see how the sodium-calcium chloride mixture can be more economical either on a cost per ton or cost-performance basis under most conditions. The mixture introduces the more expensive calcium chloride, and it introduces the costs of labor, equipment, and space for mixing operations. Furthermore, the mixture means maintaining a double inventory of chemicals, and mixtures presuppose expensive storage space and shelter costs.

As to effective de-icing, the report states that the melting rate for either straight rock salt or calcium chloride is similar above 15 F. Inasmuch as nearly all ice storms occur at higher temperatures, straight salt is better on a cost-performance basis.

At 10 F, evaporated salt was shown to be more effective than calcium chloride. Also, it is approximately one-half the price of calcium chloride.

The main weakness of the report, however, is the paucity of data. In fact, there are not sufficient data to reach satisfactory conclusions from a statistical point of view.

The curves drawn in this report reportedly show a relationship between the volume of ice melted in a given length of time for salt, calcium chloride, and salt-calcium chloride mixtures. An examination of the data used to derive these curves indicates that insufficient data were collected in the experimental portion of this work to justify the curves presented. For example, in the M test series for calcium chloride pellets, no single series of tests had more than three points that could be plotted. Test M-1-2 had only one point, tests M-3-2 and M-4-2 only two points. Because an infinite number of curves can be fitted to these data, the curves presented do not seem justified.

It is understood that some correlation of the various test conditions was attempted by assuming linear interpolation within the variables such as ice thickness, temperature and salt application. No justification is presented to substantiate that such a linear relationship exists.

The function for V (Eq. 1) could not have been developed from the data presented. The small body of data could not indicate such a function.

In closing, it should be emphasized that this report covers only the removal of ice after it is formed by the use of chemicals and chemical mixtures. Good maintenance practices indicate that chemicals be applied not after the ice is formed but during the storm to prevent just such an ice build-up.

The logical explanation for the effectiveness at lower temperatures of evaporated salt over rock salt is the absence of fines in the rock salt. It would seem desirable to suggest to the Minnesota Highway Department that they consider testing a rock salt with fines rather than recommend the use of a salt containing ⅓ calcium chloride.

The results of storage tests showing that rock salt could not be stored outdoors longer than two months should be re-evaluated in view of the fact that there are at least two anti-caking agents in use (prussian blue and yellow prussiate of soda) that are effective in keeping rock salt from caking indefinitely.

B. F. HIMMELMAN, Closure—The paper presented was prepared as a progress report covering the results of a limited number of ice removal tests run on artificially prepared ice. The study was by no means considered as a fully comprehensive test with all variables controlled or covering all possible chemical application rates. It is recognized that much more work need be undertaken to expand and verify the conclusions and recommendations indicated and this is so stated in the report.

The test data do not show that the efficacy of salt at temperatures down to 10 F to be at least equal to the mixture as stated by Mr. Jeydel. In five series of tests where the ice and air temperatures ranged from 12 to 24 F and 6 to 12 F, respectively, rock salt produced a "clear" wheelpath in only one series, whereas the mixture of ⅓ calcium chloride and ⅔ sodium chloride pellets produced a "clear" wheelpath in three of the five test series.

With reference to economy, no statement is made that a mixture is more economical than the single chemical. Location of source of supply is too great a factor when considering economy of either one or both of the chemicals as compared to a mixture of the two. The reference in the report to economy of a chemical mixture is obviously in comparison to an abrasive-chloride mixture.

Practically all ice storms admittedly occur when the temperature is at or near the freezing point; however, Mr. Jeydel assumes the temperature never drops below 15 F before the ice has dissipated. In Minnesota, the temperature quite often plunges rapidly from above freezing and it is this phenomenon that often produces the ice storm.

The curves shown in the report for the relationship between volume of the ice melted vs time for all materials tested are not direct plots of the data from any one single series of tests. They are trends based on the mean of the observed and calculated data of all the tests for one material within the temperature ranges indicated. This method of approach was necessary because of the many uncontrolled variables such as wind, temperature, and humidity present at one series of tests which were different in another series even though the temperature may have been within a very narrow range. Jeydel's examination of the data presented for the M series of tests must have been somewhat hurried because the M-1-2 series had three points (0, 15, and > 120 min); M-3-2 had two points (0 and 15 min); and M-4-2 had four points (0, 15, 45, and >75 min). Only one good curve for each of these series can be drawn through these data.

Effect of Contract Mowing on Massachusetts Maintenance Costs

ROBERT W. O'BRIEN, Assistant Highway Landscape Supervisor, Massachusetts Department of Public Works

• THE Massachusetts Department of Public Works through the Roadside Section of its Maintenance Division considers itself among the forerunners, nationally, in the establishment of contract mowing procedures and standards as a result of 10 years of experience and analysis in this method of turf control. This report summarizes the reasons for the undertaking, its history, present practices, and proof of its economic practicability as an antidote for any who find the grass "maintenance squeeze" increasingly burdensome.

BACKGROUND

Following World War II it became increasingly evident that the U.S. highway systems were inadequate and that concentration on their expansion was inevitable. As the Department's program unfolded, particularly assisted by the Federal Interstate System, with the policies of wider layouts and retention of scenic quality, the management of grassed areas loomed increasingly overpowering in the maintenance-dollar picture.

Before 1953, all grass mowing along Massachusetts highways was accomplished by Department personnel augmented by the employment of temporary summer labor and the rental of mowing equipment. This procedure was necessary as the permanent labor organization was based on the minimum required for year-round daily physical maintenance and snow and ice control activities. Also, with the exception of specialized equipment that cannot be easily rented it is not economical to own equipment unless it can be used at least 80 percent of the year. These basic concepts initiated a thorough exploration into the economic desirability of supplementing the permanent labor forces with contract methods.

Spiraling mowing expenditures in the early 1950's precipitated formulation of contract procedures. The factors evident for consideration (since proved to be the keys conducive to savings under contract) were the contractors' profit motive, their specialized experience, and ownership of the latest equipment in sufficient quantity. State forces were subject to the drawback of regulating inexperienced summer help, forced to suffer with the sporadic, often ill-timed, hiring of vitally needed equipment, and the inefficiency of being only part time on mowing because frequent occasions required their being away temporarily on other maintenance projects. Because grass growth in the warmer months is totally resistant to postponement, major relief, if not total salvation, was seen in substituting for the necessary working personnel and their administration, a minimum of directed supervision and a maximum delegation of responsibility. The latter has become greatly simplified by the development of highly effective contract special provisions. The Department has since gained decidedly in efficiency and standards as well as economy from this working procedure which is adapted to keep an even pace with the ever-expanding State highway system.

In the long quest for the most practical solution to this problem, a performance-type contract has evolved which clearly outlines the responsibility and performance expected in the special provisions (Appendix) and guarantees, under bond, satisfactory accomplishment. Each contract requires a pattern of mowing by items to include all grass growth within the full width of the right-of-way along the entire length of the contract section. This places the responsibility of determining the physical extent of the work on the contractor before bidding, and eliminates controversy in wording of job descriptions.

Paper sponsored by Committee on Maintenance Costs.

The Department's problem of supervision is reduced to a minimum because periodic inspections suffice.

The Department in 1953 awarded 14 pilot contracts for mowing along 77 miles of selected divided highways specifying 10 cuts of medians, islands, etc., under Item 1, and 5 cuts of roadsides for a minimum width of 10 feet under Item 2. These early contracts did specify reel and cutter bar use under the respective items, a stipulation since discarded, permitting the use of any type of equipment that produces a clean, sharp cut without damage to the turf. This initial venture of 14 proposals drew a total of 32 bids from 12 different contractors, and the resultant successes in economy and performance laid the ground work for development of this procedure Statewide.

In 1954 a third item of mowing was introduced to cover the area beyond the limit of Item 2 mowing, making provision for the contract mowing of all grass between right-of-way lines for the entire length of each contract. These items have since then been identified as the following:

Item 1. Lawn-Type Mowing. —Designated to accomplish fine grass mowing. Cut required to leave a stand of $1\frac{1}{2}$ in. during May, June, and October, and $2\frac{1}{2}$ in. during July, August, and September.

Item 2. Roadside Hay Mowing. —Designated to accomplish rough mowing of roadside areas. Cut required to leave a stand of 3 in.

Item 3. Hay Mowing. —Designated to accomplish hay mowing of all turf areas between the limits of Item 2 and right-of-way line. Cut required to leave a stand of 3 in.

Application of these items (Fig. 1) to conditions encountered at median, interchange, roadside areas, slopes, and at guardrail locations at fill and cut cross-sections have been through time and experience designed to accomplish the following.

Divided Highways

Item 1. Lawn-Type Mowing. —11 cuts per season. All flat, to, and including 4:1 slopes on median strips, interchange bowl areas, dividing strips at ramps, traffic islands and rotaries for a maximum distance of 30 ft from all roadways.

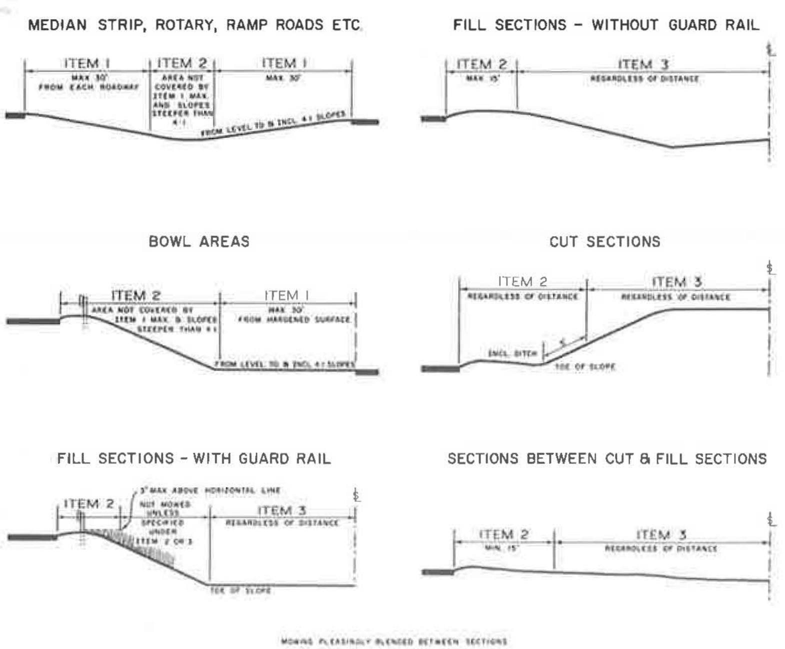


Figure 1. Application of mowing items to various topographies encountered on highway. Limits extend only to line of natural growth or areas planted for reduction of mowing.

Item 2. Roadside Hay Mowing. —5 cuts per season. The following mowing limits must be pleasingly blended:

a. All slopes from 4:1 and steeper and uncut areas beyond the maximum width of Item 1 in the previously described areas.
b. All roadside grassed areas at fill sections without guardrail for a maximum width of 15 ft as measured from the edge of grass growth nearest the road surface.
c. All roadside grassed areas at fill sections with guardrail from the edge of grass growth nearest the roadways to the guardrail and from back of the guardrail sufficiently beyond the slope crown so that no uncut grass extends more than 3 in. above a horizontal plane as extended from the ground at the guardrail.
d. All roadside grassed areas at cut cross-sections from the edge of grass growth nearest the roadway to the toe of the slope (including ditch) regardless of width, plus a 5-ft cut on the slope.
e. All roadside grass areas between cut and full sections for a maximum distance of 15 ft from grass growth nearest the roadway.

Item 3. Hay Mowing. —One cut per season. All grassed areas from the outside limits of Item 2 to the right-of-way line.

Undivided Highways

Item 2. Roadside Hay Mowing. —5 cuttings (same as preceding Item 2, b, c, d, and e).
Item 3. Hay Mowing. —One cut per season (same as preceding Item 3).

Items 2 and 3 are varied somewhat in specific locations in accordance with adjacent land use such as church, memorial, and historical sites where Item 2 may be extended beyond its normal limit to the right-of-way line.

DEVELOPMENT OF SPECIAL PROVISIONS AND CONTRACT PROCEDURES

Massachusetts, as most of New England, experiences an average grass growth of 25 in., which statistic was the basic guide in establishing the frequencies of mowing under the preceding items. After years of actual field experience, perhaps the most important consideration given to cutting schedules is the knowledge that frequent cutting is far more economical than infrequent cuttings which necessitate expensive hay pick-up operations. It is the Department's experience that grass in Item 1 and Item 2 mowing areas, if allowed to reach a height that requires a cut of over 5 in., will produce more clippings than can be decomposed without causing matting damage to turf or without the necessity of removing the clippings.

Item 1, lawn-type mowing, was established in 1953 at 10 cuttings per season. For respectable turf control, however, it was found that this had to be increased to 15 cuttings in 1954 to combat the faster growing weeds. Then, in 1957, as a result of the effective introduction of 2, 4-D weed killer, it was possible to reduce to 11 cuttings per season. The financial advantage to mowing costs realized from this material is discussed later.

Item 2, roadside hay mowing, areas are also treated with 2, 4-D weed killer to increase the standard of appearance, but this has no influence on the minimum number of annual cuttings required. Cuttings from this item were increased from 3 in 1953 to 5 in 1957 for a higher standard of roadside appearance and to eliminate the necessity of hay pick-up.

Item 3, Hay Mowing, has been maintained at one cutting, as experience has shown the importance, particularly on slope areas, of mowing grass at least once per season to prevent tufting and encourage lateral tillering and root growth for the prevention of erosion. Hay pick-up is not generally practiced on these hard-to-mow areas, as long hay clippings do not tend to mat, and, when left as a mulch material, particularly in areas adjacent to woodlands, encourage natural growth.

Item 3, originally established on a per hour basis of payment, was not conducive to a true performance contract as this required excessive inspections. In 1957, this was revised and payment was made on a per mile cut basis with all grass on both sides of the roadway cut uniformly and completed as the work progressed in one direction, thereby requiring only periodic inspection of completed miles.

Schedule of Mowing

Since 1957 the following mowing schedule has been offered in the special provisions as a guide only, and the actual number of cuttings called for to be determined by the growth rate of the grass. In the special provisions a margin for fluctuating the number of cuttings has been reserved under Item 1 in an allowable increase of one or a decrease of three. Frequency under Item 2 is a fixed standard.

The 11 cuttings under Item 1 are scheduled for three during May, two each month during June, July, and September, and one each month during August and October. The 5 cuttings under Item 2 are scheduled for one during each month of May, June, July, August, and September.

Item 3 mowing is scheduled during either July or August as directed, and is completed in conjunction with Item 2 mowing before Labor Day.

Trimming Requirements

Between 1953 and 1956, trimming requirements at guardrail locations, delineators, trees, utility poles, ledges, and other structures were varied in an effort to establish the minimum frequency necessary for sustaining a clean and neat appearance within all mowing areas.

Trimming at guardrail locations, without question, is the most difficult, time-consuming, and costly single operation involved in grass mowing, and perhaps the most essential, because a neatly trimmed and exposed guardrail has a direct influence on highway safety.

After three years review it was reluctantly concluded that a minimum of 5 guardrail trimmings per season were necessary to maintain appearance effectively.

By 1956 sufficient experience with the use of various soil-sterilant chemicals was acquired through application by Department forces, so that trimming at guardrails could be entirely eliminated from contract mowing through use of these chemicals under the entire length of a guardrail for a 2-ft strip extending from a point 6 in. in front of the posts to a point 1 ft behind.

Trimming other than around guardrail is still required under Item 1 and 2 mowing in conjunction with each cutting; however, some economies are realized through the limited use of soil-sterilant materials around structures where no damage may occur to desirable plant growth.

In 1957 all trimming was discontinued in Item 3 hay mowing areas, as expense involved in this operation on hard-to-mow, visually distant areas did not appear justified. The contractor must, however, mow as close to obstructions as possible.

Control of Mowing

The repetitive nature of this work made it apparent with the increase in number of contracts that closer control on the contractor's activities was necessary as an inducement to maintain quality of work commensurate with desirable standards.

Since 1956 the special provisions have directed the actual cutting period of each required Item 1 mowing to begin on a Monday and be completed within the week, and that Item 2 mowing begin on a Monday and be completed within two weeks.

In the event of uncut and/or untrimmed areas or any unsatisfactory cut and/or trimmed areas remaining after 5:00 PM on Saturday of said week, the inspecting engineer estimates the number of hours that would be required to satisfactorily complete the work with the use of the total normal complement of men and equipment assigned the project by the contractor. A deduction of 2 percent under Item 1 or 1 percent under Item 2 is made from the contractor's unit bid price per complete cut for each hour so estimated.

Contractors are encouraged to complete each cutting sufficiently early to allow time for correcting unsatisfactory conditions before the inspection deadline. Repeated failure to complete cuttings satisfactorily is noted in the contractor's record of performance and referred to when considered for award of future contracts.

Payment for Item 1 and Item 2 mowing is made at the contractor's lump sum bid price per complete cut on completion of each directed cutting.

As two months are allowed for the completion of Item 3 mowing, the adoption of the per mile bid unit of payment allows for partial payment of completed lengths.

Advertising and Basis of Award

As grass mowing is a seasonal operation, it is imperative that contract proposals be advertised sufficiently in advance of the mowing season to allow the prospective bidders ample time to view all projects or sites on which they are interested in bidding, and also to allow sufficient time for readvertising of a project should it be necessary to reject an original bid.

In Massachusetts where snow may cover the ground the better part of the winter and early spring, only by advertising mowing contracts in November, sufficiently in advance of snowfall, is the Department able to process all contracts well in advance of the date of the first required cutting the following spring.

Experience has shown that when a contractor has his contract in hand no later than February, he is assured of ample time to make realistic preparation for equipment, personnel, and schedule of operations sufficiently in advance of the critically important first cutting when any organizational deficiency might cause complete disruption of desired standards of work during this exaggerated growth period.

To insure complete bidding coverage for the large number of mowing proposals advertised on the same date, contractors are advised to consider each and every proposal on an individual basis, and they will not be awarded more work than their normal or reasonably expanded organization would be capable of undertaking. Under

TABLE 1

CONDENSATION OF MASSACHUSETTS ANNUAL CONTRACT MOWING PROGRAM

Year	Projects	Miles	Final Cost	Total Cost Lineal Mile Per Season	Total Bids Received	Number Of Contractors Awarded Work	Average No Of Cuts Per Contract [a]				
							Item 1		Item 2		Item 3
							Specified	Performed	Specified	Performed	Once Per Year
1953	14	77	35,346.43	$ 459.04	32	5	10	4.0	5	3.0	—
1954	30	240	155,255.75	646.89	142	12	15	12.0	3	3.0	PER HOUR
1955	16	290	211,402.02	728.97	85	11	15	11.5	3	3.3	PER HOUR
1956	20	371	252,874.48	681.60	86	13	15	13.8	4	3.8	PER HOUR
1957	25	492	307,017.83[b]	624.12	122	11	11	9.6	5	4.8	PER MILE
1958	25	529	261,002.68[b]	493.39	143	12	11	11.3	5	4.9	PER MILE
1959	33	679	263,286.68[b]				11	10.8	5	5.0	PER MILE
	20	603	117,414.27	220.92	398	27	—	—	5	4.6	PER MILE
	23	919	44,102.10				—	—	—	—	PER MILE
1960	83	2337	626,649.70[b]	268.14	310	22	11	11.0	5	5.0	PER MILE
1961	83	2337	555,777.02[b]	237.81	302	19	11	11.0	5	5.0	PER MILE
1962	86	2396	605,204.47[c]	252.58	284	15	11	11.0	5	5.0	PER MILE

[a] *Tenths reflect incompleted or unsatisfactorily completed cuttings*

[b] *Mowing - Soil Sterilant, 2, 4-D Urea*

[c] *Season in progress - only assigned available. Mowing, Soil Sterilant, 2, 4-D Urea and Maleic Hydrazide*

this procedure a contractor capable of undertaking only a few jobs may submit bids on many and have greater assurance of being successful low bidder on at least the number of jobs that he is capable of satisfactorily completing within the time allowed. At the same time, the Department has greater assurance of receiving bids on all proposals.

Bids received on a project may vary from 12 or more in centralized areas to only a few in rural areas. Fortunately, it is rare for a proposal to receive only a single bid. The annual total number of bids received (Table 1) reflects a very healthy bidding activity on this work in Massachusetts.

Reflection of Contractors Proficiency

Experience indicates that each time the Department advertised for bids for a type of maintenance work not previously done by contract, problems were created primarily because contractors were not completely familiar with the work. However, these problems were resolved mainly as an increased number of contractors indicated an interest. In this field, as in all others concerned with contract maintenance, the contractors become experienced to the point of specialization over the years, improve their methods and equipment to insure keen competitive bidding, and, from this profit drive assure the availability of a valuable pool of skilled workers for increasing economy and satisfaction.

A new low bidder must satisfy the Department that he has sufficient equipment (owned or available), personnel, and a supervisor with at least two years' experience in this kind of work before being considered for award. As few new bidders realize the actual amount of work involved in highway mowing or the many other problems to be contended with (such as traffic and travel trash litter), they are seldom awarded more than one job their first year. This policy allows both the contractor and the Department to evaluate true ability and interest in this field while minimizing the possi-

TABLE 2

CONTRACTOR BIDDING ACTIVITY

(a & b)

Contractor	1953	1954	1955	1956	1957	1958	1959	1960	1961	1962	Years of Bidding Interest
A	2	4	2	B	2	1	2	B	-	-	8
B	7	2	3	5	8	2	1	1	B	B	10
C	2	B	-	-	-	-	-	-	-	-	2
D	1	1	1	B	1	B	3	B	-	-	8
E	2	4	1	1	B	B	B	-	-	-	7
F		3	1	B	B	-	-	-	-	-	4
G		2	1	B	1	B	B	B	-	-	7
H		6	1	1	3	1	14	B	-	-	7
I		1	-	-	-	-	-	-	-	-	1
J		2	-	-	-	-	-	-	-	-	1
K		1	-	-	-	-	-	-	-	-	1
L		3	2	B	B	2	B	B	-	-	7
M		1	-	-	-	-	-	-	-	-	1
N			1	1	2	B	B	-	-	-	5
O			2	B	-	-	-	-	-	-	2
P			1	2	B	1	4	4	8	3	8
Q				1	-	-	-	-	-	-	1
R				2	B	-	-	-	-	-	2
S				2	B	3	1	1	5	B	7
T				1	2	1	B	B	-	-	5
U				1	-	B	-	B	1	-	4
V				1	B	B	-	B	-	-	4
W				1	-	B	-	-	-	-	2
X				1	B	B	B	-	-	-	4
Y					2	B	7	3	1		5
Z					2	5	10	16	18	21	6
A1					1	B	3	3	4	4	6

(a & b)

Contractor	1953	1954	1955	1956	1957	1958	1959	1960	1961	1962	Years of Bidding Interest
B1					1	5	6	13	18	24	6
C1						1	1	3	1	2	6
D1						1	4	B	-	-	3
E1						2	B	8	5	6	5
F1							1	3	3	1	4
G1							2	2	B	-	4
H1							1	B	-	-	2
I1							2	3	2	B	6
J1							1	1	1	-	3
K1							1	B	1	1	4
L1							1	2	B	B	4
M1							1	6	4	8	4
N1							1	1	1	1	4
O1							1	1	-	B	3
P1							1	B	-	-	2
Q1							1	-	-	-	1
R1							1	-	-	-	1
S1							1	-	-	-	1
T1							4	-	-	1	2
U1								3	4	2	3
V1								6	4	B	4
W1								1	B	-	2
X1								1	1	B	3
Y1								1	1	B	3
Z1										3	1
A2										3	1
B2										6	1

[a] *Numerals indicate number of contracts awarded per year.*
[b] *Letter B indicates unsuccessful bidding activity*

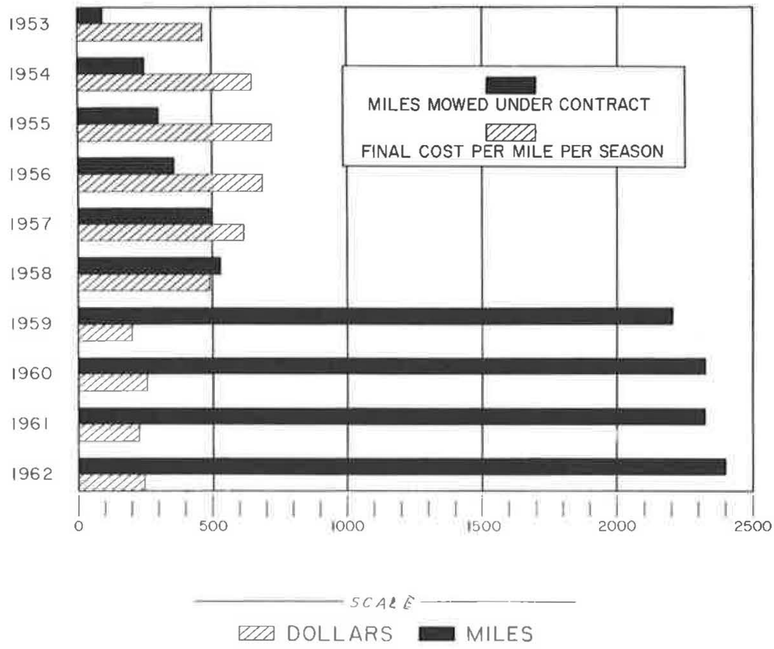


Figure 2. Annual expansion of highway miles under contract mowing program and the final cost per mile per season.

TABLE 3
1962 ROADSIDE ACREAGE RECORD

	Median Strip, Rotary etc.	Roadside	Interchange	Total	Percent
Item I Mowing	966.2	14.3	487.7	1,468.2	7.3%
Item II Mowing	378.6	5,404.3	754.3	6,537.2	32.4%
Item III Mowing	23.4	2,404.8	206.7	2,634.9	13.0%
TOTAL MOWING ACRES	**1,368.2**	**7,823.4**	**1,448.7**	**10,640.3**	**52.7%**
Non-Grass Areas (Natural Growth, Ledge, etc.)	857.5	7,818.0	875.0	9,550.5	47.3%
GRAND TOTAL ACRES	**2,225.7**	**15,641.4**	**2,323.7**	**20,190.8**	**100.0%**

bility of contractor failure. In 10 years of contract mowing, only one contractor has failed to the point that the Department had to undertake the work and assess all costs against the bonding company.

After satisfactorily completing a contract and having become familiar with the entire procedure of work, supervision, and payment, a contractor is considered in the future for award of as many contracts as his equipment and organization are capable of handling.

Record of contractor's activities (Table 2) is kept only after award of initial mowing contract. This record reflects the development of contractors in this field, indicating by year their initial award and subsequent bidding or award activity. Awards are not entirely based on low bids, as occasionally a second or third low bidder must be reached when a lower bidder has been awarded all the work he is capable of undertaking and completing satisfactorily within the mowing season.

Beginning in 1953 with the original five successful bidders, this record has expanded by an annual increase of at least three new successful bidders to today's total of 54 individual contractors who have been awarded one or more contracts. Though many of the contractors may not submit successful low bids each succeeding year, only 8 have failed to submit bids at least one year following initial award. Interest and contractor development over the years have resulted in at least 20 experienced mowing contractors bidding this work each year since 1960.

COST ANALYSIS

Reduction in Annual Cost per Mile

Although contract mowing economics reflected in a unit of cost per mile does not account for the amount of work performed in any given mile, it is, nevertheless, a reasonable barometer indicating the influence of the development of special provisions, contractors, and allied activities which contribute to over-all grass maintenance.

In 1953 late award of the initial 14 contracts covering 77 miles of highway, and during severe summer drought, reduced the number of accomplished cuttings under reel-type mowing from the specified 10 to 4, and under cutter bar mowing from 5 to 3. The total final cost of this work was $35,346.46 or $459.04 per mile per season.

On completion of this mowing, the Department conducted an extensive study on the comparative costs of mowing by contract and force account. At that time, figures indicated that the average force account cost per mile per season would be $613.17 for the work accomplished by contract at $459.04. Total force account mowing costs were estimated in the vicinity of $1,000,000 per year in Massachusetts.

Contract mowing was expanded in 1954 to cover 240 miles of highway under 30 contracts. Work performed included 12 Item 1 cuttings, 3 Item 2 cuttings and all Item 3 mowing at a final cost of $155,255.75 or $646.89 per mile per season, representing an increase of only $187.85 more per mile per season than that paid for the very limited amount of work performed in 1953.

Favorable operational results, and cost evaluations and interest shown by contractors encouraged annual expansion of this program until in 1958 all mowing on the then total of 529 miles of major highways was accomplished under 25 contracts at an ultimate cost of $261,002.08 or $493.39 per mile.

This represented a seasonal reduction of $153.50 per mile over the 1954 costs, reflecting many improvements gained through the development of both the special provisions and contractors. However, the greatest influence on this economy was the introduction of weed killer and soil sterilant chemicals to mowing areas in the 1957 season.

Beneficial Contribution of Applied Chemicals

Weed killer 2,4-D applied to all Item 1 mowing areas allowed a reduction in the number of annual cuttings required under this item from 15 to 11 without sacrifice of appearance. This operation, since developed as an annual contract maintenance function, includes the combined application of urea 45 percent nitrogen fertilizer for the rejuvenation of turf areas chemically weeded. The annual cost of this combination spray is approximately $12.00 per acre for two treatments per season. The elimination of a minimum of three Item 1 mowings represents a definite saving of at least $15.00 per acre per season. Other acreage costs are discussed later. This saving is further extended. After two successive years of treatment, areas are placed under a biennial spray program.

Also in 1957 the use of soil-sterilant chemicals applied to the entire length of all guardrails in contract mowing areas allowed the Department to eliminate the necessity of trimming at these locations. Contract application methods have also been designed for this operation, which today is extended to all guardrails on State highways under a three-year rotation program.

The economy of this operation cannot be overemphasized in its dollar savings value to the maintenance budget. A single soil-sterilant application, which effectively controls all vegetative growth for a minimum of two years, costs approximately $45.00 per mile of guardrail, represents less than one-tenth of the minimum cost of accomplishing hand trimming of an equal area for the same period.

Though the application of 2, 4-D weed-killer, urea fertilizer, and soil sterilants provide desired results for at least a two-year period, reduced to an annual consideration, the expenditure of approximately $27,000 for materials and application of these chemicals has resulted in a spectacular savings of $220,000 to over-all grass maintenance costs.

In 1959 due to the high standards attained and the economies realized, this method of turf management was expanded to cover the entire State highway system through inclusion of all nondivided highways, which contain comparatively fewer grassed areas requiring only Item 2 and 3 mowing.

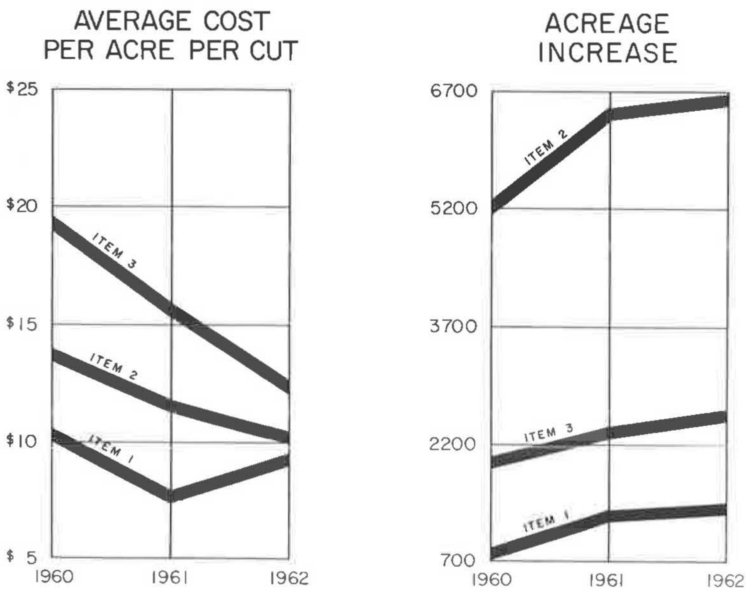
Through 1959, contract proposal assignments were for lengths on specified auto routes, and the Department was faced at that time with the complicating feature of disagreement between connecting contractors as to responsibility at intersections and grade separations. In 1960 proposals were revised to coincide with the geographical limits of the established area of a foreman's section, and this clear definition of the highway facilities contained therein precluded any overlapping of responsibility. This provided considerable supervisory advantage. Even though work is being accomplished by contract method, the section foreman must exercise interest in all physical maintenance work being performed within his individual area of reponsibility. His familiarity with this work and his daily observations greatly assist the engineer in over-all inspection. Use of this relatively static quantity facilitated and improved bidding. Contractors over the years could more easily familiarize themselves with the demands of a particular proposal area, solidifying their judgment in bidding, which since 1959 has averaged $245.00 per mile per season, representing an annual budget of approximately $600,000 for the mowing of all grass on the entire State highway system.

Final cost per lineal mile figures (Table 1) beginning with 1957 represent the total annual sum of all activities (contract and hired equipment) contributing to the maintenance of grass within contract areas. Before this date, force account maintenance records were not sufficiently detailed to segregate costs accurately, although analysis of past expenditures reveals that more than one-fourth of the total physical maintenance budget is needed for roadside work; and that approximately one-third of this is absorbed by grass maintenance activities.

Comparison of Savings Based on Acreage Analysis

As analysis based on lineal miles (Table 1) and (Fig. 2) does not provide the unit of accuracy necessary for segregated roadside activity costs, a record of all acreage on State highway layout, exclusive of hardened surface, shoulder and bridges, has been maintained since 1960. This record (Table 3), annually revised to absorb changing land use and new mileage, presently totals 20,190.8 acres of roadside layout, of which 10,640.3 or 52.7 percent are grassed areas, normally requiring annual maintenance.

Of this total 20,190.8 acres, 1,468 or 7.3 percent are Item 1 mowing areas, 6,537.2 or 32.4 percent are Item 2 mowing areas, and 2,634.9 or 13.0 percent are Item 3 mowing areas. The remaining 9,550.5 acres or 47.5 percent of the total layout are either non-grassed areas such as ledges, where natural growth exists, or planting has been accomplished for the reduction of mowing.



Acres = 7,829 10,045 10,620

Figure 3. Three-year analysis of contract mowing per acre per cut.

TABLE 4

THREE YEAR ANALYSIS OF CONTRACT PER ACRE PER CUT[a]

YEAR	ACREAGE	COST PER CUT
ITEM 1		
1960	773	$10.13
1961	1,296	7.79
1962	1,393	9.08
ITEM 2		
1960	5,149	$13.11
1961	6,362	11.50
1962	6,597	10.26
ITEM 3		
1960	1,907	$19.12
1961	2,387	15.81
1962	2,630	12.66

(a) *Contract bid prices per complete cut of Item 1 and Item 2 and bid price per mile of Item 3 converted to cost per cut of existing grassed acreage within Item areas.*

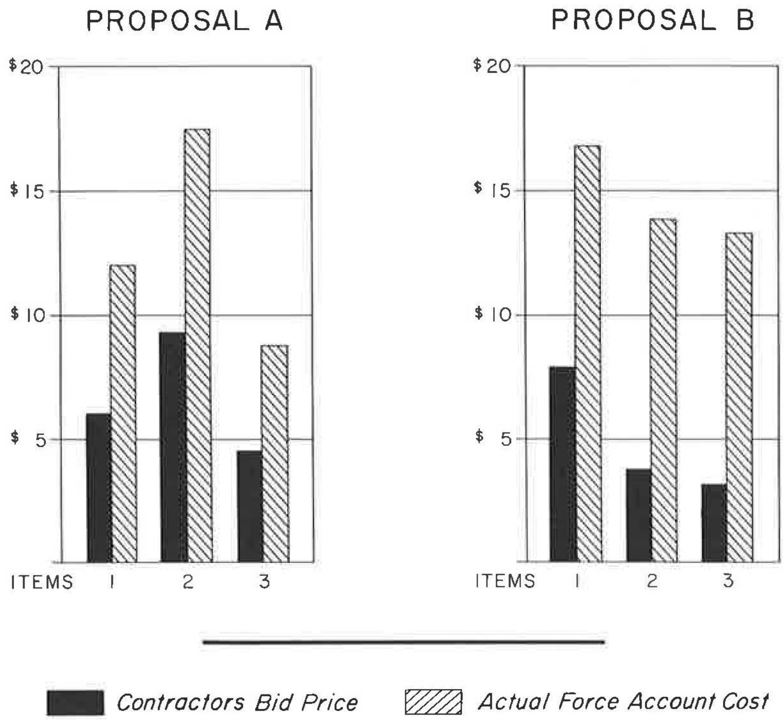


Figure 4. 1961 comparative cost per acre per cut study of contract vs force account mowing.

TABLE 5

CONTRACT VS FORCE ACCOUNT COMPARATIVE COST STUDY ANALYSIS OF TOTAL COST PER ITEM

PROPOSAL A (16.2 MILES)

Item	Force Account Cost	Contract Bid Price	Acres
1	$ 7,878.23 (64.8%)	$ 4,200.00 (64.3%)	93.7
2	2,934.58 (24.2%)	1,600.00 (24.5%)	8.6
3	1,348.16 (11.0%)	729.00 (11.2%)	150.1
	$12,160.97 (100 %)	$ 6,529.00 (100 %)	252.4

PROPOSAL B (23.4 MILES)

Item	Force Account Cost	Contract Bid Price	Acres
1	$ 5,554.53 (38.7%)	$ 2,600.00 (51.4%)	40.9
2	7,596.19 (52.9%)	2,150.00 (42.5%)	111.4
3	1,203.84 (8.4%)	312.00 (6.1%)	92.3
	$14,354.50 (100 %)	$ 5,062.00 (100 %)	244.6

Force Account figures represent actual costs of accomplishing work. Contract bid price represents what work would have cost if accomplished by contract method.

The geographical division of the total 10,640.3 grassed acres is 1,368.2 of medians, 1,448.7 of interchanges and 7,823.4 of roadsides.

A single mowing of all grass within an average mile representing 4.5 acres or a width of 40 feet is being accomplished this year at a contract cost of only $48.11 per mile.

A study of acreage data applied to the unit bid prices received for each item of mowing on all contracts for a three-year cost per acre per cut analysis (Fig. 3) further substantiates the economic necessity of this work being performed under competitive contracting.

Since 1960 (Table 4) the Item 1 cost per acre per cut has been reduced from $10.13 to $9.08; Item 2 cost per acre per cut from $13.11 to $10.26; and Item 3 cost per acre per cut from $19.12 to $12.66.

During this three-year period 2,791 acres of mowing area were added to maintenance responsibility, and the Statewide contract mowing cost has been reduced by more than $20,000.

Special Study of Contract vs Force Account Mowing Costs

By 1961 this Department's contract mowing procedures had been developed until it was felt that these methods effected a sound turf management program under which a high degree of both turf protection and pleasing turf appearance was achieved.

The standard of maintenance of grassed areas was at a point where it was again desirable to obtain accurate comparative costs between contract and force account mowing, and following receipt of bids for the 1961 contract mowing, two proposals, selected at random, were rejected so that the work might be accomplished by force account methods.

To insure that force account cost data, accumulated throughout the season, be comparable with contract cost figures, all contract mowing special provisions were followed.

Only costs normally related to the contractor were considered, and charges normally absorbed by the Department under contract mowing, such as raking and pickup of the area, hay pickup when necessary, final inspection of each cutting, engineering and clerical overhead charges, were not included in the charges against this work.

At the completion of this work, detailed weekly reports of force account mowing costs were analyzed on the basis of work units accomplished in comparison to prices that would have been paid to the low bidder of each project at the unit price submitted.

Reduced to a seasonal cost per mile, the contract price of $292.70 was comparable with the current average statewide contract cost, whereas the actual per mile cost of $669.58 for accomplishing this work by force account methods was comparable to the analysis of force account costs made in 1953.

Both study areas were under normal programing of weed killer, fertilization, and soil-sterilant spraying, although their costs are not represented in the mowing analysis figures.

Figure 4 shows the similarity between the contractor's evaluation of costs per acre per cut of each item and the graduated Department force account costs on both projects.

Problems encountered in force account performance of this work were basically the reason for this Department's original exploration into the feasibility of performing this work by contract method back in the early 1950's.

Results of this comparative cost study (Table 5) substantiate the analysis of mowing costs made before doing this work by contract, and show that the over-all costs of grass mowing have been, through this method, reduced by at least 50 percent, while at the same time providing a uniform standard of mowing and a far more uniform and clean roadside appearance throughout the State highway system than had previously been accomplished with force account mowing.

CONCLUSION

Value of Standards

The desirability of a standard for grass mowing has been apparent to the AASHO Committee on Maintenance and Equipment for some time, and through the efforts of its subcommittee on Highway Maintenance Standards, a set of standards developed as a guide for roadside mowing was approved and ordered printed as a guide by the Executive Committee on June 19, 1962. These guide standards are very similar to those under which the Department's contract mowing procedures have been developed.

The true values in accomplishing many roadside activities by contract method under such standards go beyond budgetary economics in that an efficient high standard of roadside maintenance produces the uniformity, pleasing appearance, and safety that is expected by the general public of today's modern highway system.

Summary of Influence in Mowing Economy

The pursuit of economy in grass maintenance is an expanding and never-ending challenge. The Department is presently engaged in its second year of contract application of growth inhibitor maleic hydrazide to 2,790 acres of Item 1 and Item 2 mowing within 19 mowing contract areas. Through the use of this chemical, the number of mowings called for within these areas under Item 1 have been reduced from 11 to 4 and under Item 2 from 5 to 2. This undertaking, in its infancy as contract mowing was in 1953, is costing more per acre treated than would a full schedule of mowing, reflecting the present high cost of the chemical in comparison to the low prices being paid for contract mowing. As optimism is the essence of progress, the additional expense is felt to be more than justified, as knowledge gained and refinement of procedures may place at the Department's disposal another tool whose potential influences on future maintenance savings are today not fully understood.

Although the title of this paper indicated a discussion of only contract mowing, the influence of chemical turf control is so inextricably entwined with the advantages from contract mowing in the Massachusetts program, it was necessary that its features be included as well to present a true picture of all influences contributing to the overall economy of the grass maintenance program.

As long as the contracting industry shows competitive interest as favorably as today, it is the intent of the Massachusetts Department of Public Works to continue mowing all grass on the State highway system by contract method. It is hoped that this brief synopsis of the Department's efforts has highlighted the tremendous savings made possible through the development of contract procedures.

Appendix

SPECIAL PROVISIONS

FOREWORD

The work to be done hereunder shall conform to the Massachusetts Department of Public Works Standard Specifications for Highways and Bridges 1953 Edition, the Amendments, Addenda, Uniform Special Provisions and Supplementary Uniform Special Provisions for Bridges included in a Pamphlet entitled "Amendments and Addenda, June 1961," and the Special Provisions included hereinafter. The said Standard Specifications were approved by the Board of Commissioners on January 26, 1954.

Such other Amendments and Addenda, as may be included herewith, are hereby specifically made a part of the Special Provisions of this contract as fully and to the same effect as if they had been set forth at length herein.

References in the following Special Provisions, unless otherwise stated, are to the aforesaid "Standard Specifications" and "Amendments and Addenda." In case of conflict between these Special Provisions and the aforesaid Standard Specifications, Amendments, Addenda, Uniform Special Provisions, and Supplementary Uniform Special Provisions, these Special Provisions shall take precedence and shall govern.

The enforcement of the requirements of any of these Special Provisions shall not be construed as waiving any of the rights of the Party of the First Part contained in any of the other provisions of this contract.

PROCUREMENT OF LABOR (Supplementing Article 63)

Attention is directed to the assistance which is available from the State employment services in the recruitment of workers in all occupations and skills. Contractor may obtain assistance in such employment of labor first from the local labor market and, when the required workers are not so available, he may obtain such assistance through the nation wide clearance system of the U.S. Employment Service.

CONSTRUCTION STAKINGS (Supplementing Article 34)

All the requirements of Article 34 as amended regarding employment by the Contractor of engineering personnel or the furnishing and setting of stakes by him are hereby waived by the Department for this project. The Department will furnish all engineering necessary to maintain lines and grades and accurate control for the work.

CLASS I BITUMINOUS CONCRETE PAVEMENT TYPE I-1, shall be constructed as required in accordance with the Plans, the relevant provisions of Section B-18 as revised April, 1961 and included in Pamphlet entitled "Amendments and Addenda," June, 1961.

Compensation for this work will be made at the contract unit price per ton under Item B18-1.

JUNE 1961

SPECIAL PROVISIONS

for

MOWING GRASSED AREAS ALONG STATE HIGHWAYS

SCOPE OF WORK

The work to be done under this contract shall consist of mowing all grassed areas as outlined in these special provisions located within the State Highways right-of-way layout including street approaches between such points or stations as are designated in the itemized proposal included hereinafter.

All interchange areas to the Limits of all adjoining ramps within the State Highway Layout, between the described Limits of each route as indicated on the Proposal page, shall be included in their entirety, unless specific exclusion is noted. The intersecting State Highway from the Limits of all adjoining ramps in both directions, to the end of the State Highway Layout shall also be included in all instances where the intersecting State Highway has not been described in some mowing proposal. Special conditions relating to individual projects are also outlined on the itemized proposal pages.

Information relative to Limits of the State Highway Layouts involved may be obtained at the Department's District Offices, the location of which are listed following these Special Provisions.

The work shall be done in accordance with these Special Provisions and the relevant provisions of the Department's General Requirements and Covenants included in Division I of the Standard Specifications for Highways and Bridges, 1953 Edition.

All roadside, median strip, bowl, dividing strip, rotaries, and/or traffic islands, grassed areas shall be completely mowed under either Items 1, 2 and/or 3 as follows irrespective of previous mowing treatment.

MEDIAN STRIPS:

A. All flat or rolling slopes from level to and including 4 to 1 slopes shall be mowed under Item 1 for a maximum width of 30 feet from the edge of the hardened surface of each roadway.
B. All moderately steep slopes from 4 to 1 to and including 2 to 1 slopes and those uncut areas of slopes flatter than 4 to 1 not covered by the maximum width for Item 1 mowing shall be mowed under Item 2.
C. All slopes steeper than 2 to 1 shall be mowed under Item 2.

BOWL AREAS (areas completely enclosed by roadways at interchanges:)

A. All flat or rolling slopes from level to and including 4 to 1 slopes shall be mowed under Item 1 for a maximum width of 30 feet from the edge of the hardened surface of the roadway.
B. All moderately steep slopes from 4 to 1 and including 2 to 1 slopes and those uncut areas on slopes flatter than 4 to 1 not covered by the maximum width for Item 1 mowing shall be mowed under Item 2.
C. All slopes steeper than 2 to 1 shall be mowed under Item 2.

DIVIDING STRIPS AT RAMPS, TRAFFIC ISLANDS AND ROTARIES:

A. All flat or rolling slopes from level to and including 4 to 1 slopes shall be mowed in their entirety under Item 1.
B. All moderately steep slopes from 4 to 1 to and including 2 to 1 slopes shall be mowed under Item 2.
C. All slopes steeper than 2 to 1 shall be mowed under Item 2.

Median Strips, Bowl Areas, Dividing Strips at Ramps, traffic island and rotaries shall be mowed as outlined above regardless of the presence of guard rail.

All grassed areas regardless of width, in the above described locations and whether continuous in topography or not shall be mowed under either Item 1 or 2. This applies equally to cut sections, fill sections and sections between cut and fill.

ROADSIDES shall be mowed as follows:

Only Items 2 and 3 will be required on roadside grassed areas except at such locations as are outlined on the Proposal page.

AT FILL CROSS SECTIONS

1. With Guard Rail:
   a. From the edge of grass growth nearest the road surface to the guard rail and from back of the guard rail extending away from the road surface to a point sufficiently beyond the break of the slope crown so that no uncut grass on the slope will extend higher than 3-inches above, a horizontal plane as extended from the ground at the guard rail shall be mowed under Item 2.
   b. The balance of the fill slope area will not be mowed under either Item 2 or 3 unless specific sections are so specified on the itemized proposal sheet.
   c. Grassed areas from the toe of the fill slope to the layout line shall be mowed under Item 3.
2. Without Guard Rail:
   a. For a maximum width of 15 feet of grassed area as measured from the edge of grass growth nearest the road surface shall be mowed under Item 2.
   b. Grassed areas from the outside limits of Item 2 mowing to the layout line shall be mowed under Item 3.

AT CUT CROSS SECTIONS

a. From the edge of grass growth nearest the road surface to the toe of the slope (including ditch) plus a width of 5 feet on the slope regardless of the total width involved shall be mowed under Item 2.
b. Grassed areas from the outside limits of Item 2 mowing to the top of the slope shall be mowed under Item 3.
c. Grassed areas from the top of the slope to the layout line shall be mowed under Item 3.

AT CROSS SECTIONS BETWEEN FILL AND CUT SECTIONS

a. From the edge of grass growth nearest the road surface for a minimum distance of 15 feet shall be mowed under Item 2. (This mowed area must blend with areas mowed at fill and cut cross sections as described above).
b. Grassed areas from the outside limits of Item 2 to the layout line shall be mowed under Item 3.

Each foot of length on each project will be either in a fill Cross Section, a cut cross section or a cross section between fill and cut sections. Usually a cross section at any particular point extends from the layout line on one side of the road to the layout line on the opposite side of the road. However, as it relates to roadside mowing, each side of the center line of the involved roadway shall be considered separately.

The section shall be considered a CUT cross section from the center line of the roadway to the layout line where the side slope nearest the layout line has been constructed below the original topography of the ground.

The section shall be considered a FILL cross section from the center line of the roadway to the layout line where the side slope nearest the layout line has been constructed above the original topography of the ground.

Regardless of the foregoing limitations grassed areas within the limits of Rest Areas and for a distance not to exceed 15 feet outside the limits of the Rest Area, shall be mowed under Item 2. This shall also apply to truck turnouts.

All other grassed areas not outlined above shall be mowed under Item 3 with the exception of certain slopes on which desirable natural growth, as identified by the District Highway Engineer, has established itself; vines, seedlings and ground cover plants have been established for erosion control; or where mulch has been applied for erosion control and the inducement of natural growth. In such areas the Contractor shall mow only the clear grass area up to the line of natural growth, and/or erosion control plantings, and mulch. Areas shall be mowed where mulch has been applied for the inducement of grass growth.

The Contractor shall not be required to rake or pick up any cut grass.

This contract does not require trimming at guard rail locations, since the total length of guard rail sections from a point 6-inches in front of the posts to a point 1 foot behind the posts will be treated by the Department with chemicals which will render the soil sterile thereby eliminating all vegetative growth. Also this contract does not include trimming in areas mowed under Item 3 but the Contractor shall mow as close to all obstructions as possible.

Neat trimming will be necessary in all areas mowed under Items 1 and 2 around poles, trees, ledges, delineators, utility poles, curbs, piers, abutments and other structures coming within the item area and shall be conducted simultaneously with the mowing during each cutting operation. All curbing shall be trimmed and exposed.

Grassed areas which are saturated with water during certain periods of the year to the point where equipment may not be used without extensive damage to the turf shall not be mowed at the particular time but shall be mowed under the applicable item when dry.

Certain newly seeded areas on new Construction may not require a full schedule of mowing until the establishment of a stable turf.

Certain areas within the limits of locations described on the Proposal page may be under contract for construction or reconstruction and may not require a full schedule of mowing.

The foregoing outline of mowing treatment shall apply to both sides of the specified roadways and the outside areas of interchange, exit and access roadway locations, as well as all median strips, bowl areas, dividing ramps, rotaries and traffic islands.

Grass referred to under this contract shall include areas that consist of all grass, part grass and part succulent weed growth or all succulent weed growth present within the State Highway layouts. Woody growth or brush shall not be classified as grass.

The mileage as indicated on the Proposal page is the approximate number of base line horizontal miles, in either direction, between all described limits of the contract regardless of whether single or double barrel roadway is involved and is furnished as a guide only.

This mileage does not reflect the mileage involved in ramp and approach roadways at interchanges and intersections, rotaries, turnouts, etc., even though work may be required in these areas under these Special Provisions.

Prosecution of Work

No work shall be done under this contract on Sundays or Holidays.

The Contractor shall begin work under the various items and prosecute same in accordance with these Special Provisions and as directed by the District Highway Engineer whose authority is outlined in Article 28 of the Department Standard Specifications.

During the months of MAY, JUNE and OCTOBER, each directed cutting under Item 1 shall be performed in such a manner that the result will provide a stand of mowed grass $1\frac{1}{2}$ inches tall immediately following cutting, and during the months of JULY, AUGUST and SEPTEMBER, in such a manner that the result will provide a stand of mowed grass $2\frac{1}{2}$ inches tall immediately following cutting.

As the actual total number of cuttings necessary under Item 1 shall be determined by the growth rate of the grass to be mowed, the District Highway Engineer may direct that the number of cuttings under proposals calling for eleven cuttings be reduced by a maximum of three or that the number of cuttings be increased by a maximum of one, or that under proposals calling for three cuttings the number of cuttings be increased by a maximum of one.

The actual cutting period of each required mowing under Item 1 as specified on the Proposal Page shall be as directed and shall begin on MONDAY and be completed within the week specified.

The first cutting under Item 1 on proposals calling for eleven cuttings shall not begin before the last MONDAY in APRIL and may be delayed until the following week if so directed by the District Highway Engineer. Under such proposals only one cutting under Item 1 shall be made during the month of OCTOBER and shall commence no earlier than the first MONDAY in OCTOBER.

Under proposals calling for only three Item 1 cuttings, such cuttings shall be scheduled as directed by the District Highway Engineer. The tentative schedule appearing in these Special Provisions may be used as a guide.

As the necessary frequence of the number of cuttings of Item 2, as called for on the Proposal Page, shall be determined by the growth rate of the grass to be mowed, the period of each mowing shall be as directed by the District Highway Engineer.

Each cutting under Item 2 shall be performed in such a manner that its results will provide a stand of mowed grass 3 inches tall immediately following cutting.

The prosecution of work, as described under Item 2, shall apply to both proposals calling for five and for two Item 2 Mowings.

Item 3 mowing shall commence at one end of the contract mowing area as directed, and proceed in one direction for the entire length of the project. Item 3 mowing shall be accomplished during either JULY or AUGUST as directed, and shall be completed before Labor Day. Under proposals calling for five Item 2 Mowing, the Item 3 Mowing shall be accomplished in conjunction with Item 2 Mowing. Only those lengths which have been completed and satisfactorily mowed on both sides of the road shall qualify for partial payment under this item.

All cuttings under Item 3 shall be performed in such a manner that its result will provide a stand of mowed grass 3 inches tall immediately following cutting.

Under no circumstances shall the Department be responsible for any damage to the Contractor's equipment due to any obstacles, (stone, sand, debris, etc.) he may encounter during the work to be performed under this contract and the Cpntractor shall not receive any compensation therefor, in addition to the contract unit price per cutting.

All trimming work, (power, and/or hand equipment), necessary under Item 1 and Item 2 shall be conducted simultaneously with each mowing, and the Contractor will be required to organize his operations accordingly.

The Contractor shall be fully responsible for the supervision and the satisfactory performance of his organization. The Contractor shall be responsible for the completion of all work called for under this contract, as set forth in these Special Provisions and as directed, in a manner satisfactory to the District Highway Engineer.

During the period of mowing operations, the Contractor shall consult the Engineer for inspection and tentative approval of work being accomplished, so that in the event of unsatisfactory work, sufficient time will be available to the Contractor, for remowing such areas in order that the total cutting may be completed in a satisfactory manner within the time specified.

Upon completion of each Item 1 and/or Item 2 Mowing, and on completed lengths of Item 3 Mowing, the Contractor shall notify the Engineer so that a final inspection of the involved area may be made before approval of the item of work.

Repeated failure of the Contractor to perform satisfactorily the work as called for under this contract within the specified periods shall be deemed sufficient reason for the Department to protect its interests in accordance with Article 76.

The following cutting schedules for contracts calling for eleven Item 1 and five Item 2 mowings are tentative guides only as the actual cutting periods will be as directed ed by the District Highway Engineer:

ITEM 1	MONTH	NUMBER OF CUTTINGS	ITEM 2	MONTH	NUMBER OF CUTTINGS
	MAY	3		MAY	1
	JUNE	2		JUNE	1
	JULY	2		JULY	1
	AUGUST	1		AUGUST	1
	SEPTEMBER	2		SEPTEMBER	1
	OCTOBER	1			

The following cutting schedule for contracts calling for three Item 1 and two Item 2 mowings are tentative guides only as the actual cutting periods will be as directed by the District Highway Engineer:

ITEM 1	MONTH	NUMBER OF CUTTINGS	ITEM 2	MONTH	NUMBER OF CUTTINGS
	MAY	1			
	JUNE	1		June	1
	SEPTEMBER	1		SEPTEMBER	1

If in the opinion of the Engineer, it is necessary at any time in order to maintain the schedule of cuttings under each item, the Contractor shall, when directed, employ such forces and equipment for one or more additional shifts as will be required to take full advantage of all daylight hours to insure the proper completion of the work. The Contractor shall not receive any compensation therefor in addition to the contract unit prices.

Note: It is the intention of the Department to treat all roadside, median strip, ramp road, and traffic island grassed areas, under contract mowing, for a width of 20 feet with weed control chemicals for the elimination of such weed growth. Two applications of chemicals are planned for the season and as these areas may not be mowed for 72 hours before treatment nor 48 hours after treatment it will be necessary for the District Highway Engineer to coordinate these two operations so as to insure the least inconvenience to all parties concerned.

Examination of Location

The Contractor must satisfy himself by his own investigation and research regarding all conditions affecting the work and the amount of work to be done, the labor and equipment needed, and make his bid in sole reliance thereon.

Rejection of Proposal (Supplementing Article 10)

Proposals containing abnormally low unit prices for Items 1 or 2 and/or 3 thereby making profitable the potential failure to complete portions of the work in lieu of payment of the specified credit, shall be rejected as informal.

Competency of Bidders (Supplementing Article 12)

As this contract contains work of a special nature, Contractor to whom the contract will be awarded may be required to furnish the Department with a written statement, indicating that he has the necessary skill, experienced personnel and a qualified supervisor who has had at least two years experience in this kind of work together with a listing of all equipment of his own and equipment available to him which he intends to use in performing the work required under this contract in a satisfactory manner and within the time stipulated.

The low bidder on each proposal will be required to give evidence that he can meet the requirements of Article 12.

A Contractor who is low bidder on more than one proposal will be required to give evidence that he can meet the requirements of Article 12 for all projects collectively on which he is low bidder so that work on all projects will be carried on simultaneously when necessary to complete the work within the specified time.

Consideration of Bids (Supplementing Article 14)

This proposal book contains a number of separate projects for the same type of work.

Prospective bidders are advised to consider each and every project on an individual basis.

The Department reserves the right to waive any informality in or reject any or all proposals; therefore, low bidders will be considered for award on the basis of bid price, performance record, experience, organization, equipment, etc.

An award will not be made to a Contractor who is not equipped to undertake and complete the work within the specified time.

INSURANCE REQUIREMENTS (Supplementing article 46)
The limits of the several kinds if liability insurance required for this contract are listed as follows:

Public Liability	\$25,000/\$75,000
Property Damage Liability	\$25,000/\$50,000
Protective Public Liability	\$25,000/\$75,000
Protective Property Damage Liability	\$25,000/\$50,000

Attention is directed to amendment to said Article 46 wherein it stipulates that the insurance shall cover all damages to property whether above or below ground.

PROTECTION AND RESTORATION OF PROPERTY (Supplementing Article 55)
The Contractor shall, at his own expense, preserve and protect from injury all property either public or private along and adjacent to the roadway, and he shall be responsible for and repair at his own expense any and all damage and injury thereto, arising out of or in consequence of any act or omission of the Contractor or his employees in the performance of the work covered by the contract prior to completion and acceptance thereof.

The Contractor shall be held liable for all damage done to signs, delineators and all turf areas, desirable natural growth as identified by the District Highway Engineer, seeding shrubs and trees by his equipment and personnel. Damages shall include among other things; skiving, scraping or gouging of trees, shrubs and turf areas: ruts and deep wheel depressions on turf areas; and ruts, deep wheel depressions and wheel slipping damage on slope areas.

Certain areas to be mowed may contain survey stakes which must be preserved. Mowing will be required to within one (1) foot of said stakes but no trimming will be required around them.

EQUIPMENT
The Contractor will be required to furnish equipment that will perform work satisfactorily and shall have before award of any contract sufficient equipment of his own, or furnish proof of its availibility to him, which will satisfactorily complete all work called for under this contract.

Equipment necessary for mowing and trimming under this contract may consist of: tractor operated reel, rotary and sickle bar grass cutting machines; power driven walk behind reel, rotary and sickle bar grass cutting machines; power driven hand operated grass cutting machines; manually operated grass cutting machines and hand operated grass cutting tools and any other grass cutting equipment, machines or tools.

Machines referred to in these special provisions are to be considered as any type of equipment applicable for use in mowing and trimming all grass within the limits of the contract area as specified in these special provisions and as directed by the District Highway Engineer.

The equipment furnished by the Contractor must be in good repair and shall be maintained so as to produce a clean, sharp cut to the grass at all times.

Equipment which in any way pulls or rips grass or damages the turf shall not be allowed to operate under this contract. If in the opinion of the District Highway Engineer the Contractor has insufficient equipment of any type on the job to satisfactorily complete the work under the various items within the time specified, the Contractor shall provide additional equipment, as directed by the District Highway Engineer.

NOTE:
The Contractor is reminded that reference to mowing in these Special Provisions does not refer exclusively to that grass cutting which can be accomplished by tractor drawn equipment. Hand and/or other power equipment must be used if necessary to satisfactorily complete the work.

SAMPLE

Mowing Grass
M-63-12

Example of Special conditions as would be noted on a proposal page under any given Route or Town description.

(Extending Item 2 beyond normal limits)
Note: On Route 128, Needham, from Station 123+00 at the Kendrick Street Overpass, northerly to Station 175+00 approximately 600 ft., north of the R.R. overpass, on the northbound barrel shall be mowed under Item 2 from the grass growth at the edge of the outer roadway to the layout line regardless of the presence of guard rail.

(Entire Proposal Item 1 & 2 areas treated with Growth Inhibitor MH-30. Item quantity would call for three Item 1 and two Item 2 cuts).
Note: All Item 1 and Item 2 grassed areas within the limits of this proposal have been or are to be treated with Maleic Hydrazide (MH-30) and as this chemical inhibits grass growth, it should be noted that the proposal calls for a reduced number of mowings.
Cuttings shall be made by the Contractor only when directed by the District Highway Engineer.

To the Party of the First Part:

The undersigned, as bidder, declares that the only persons or parties interested in this proposal as principals are those named herein; that this proposal is made without collusion with any other person, firm or corporation; that he has carefully examined the location of the proposed work, the proposed form of contract, the standard specifications and plans therein referred to and the Special Provisions hereto annexed; and he proposes and agrees, if this proposal is accepted, that he will contract with the Party of the First Part, in the form of the contract referred to herein and to be annexed hereto, to provide all necessary machinery, tools, apparatus and other means of construction, and to do all the work and furnish all the materials specified in the contract, in the manner and time therein prescribed, and according to the requirements of the Engineer as therein set forth, and that he will take in full payment therefor the following unit prices, to wit:

ITEM NO.	QUANTITY	ITEM WITH UNIT BID PRICE WRITTEN IN WORDS	UNIT PRICE		AMOUNT	
			DOLLARS	CENTS	DOLLARS	CENTS
1	11	Cuttings of LAWN TYPE MOWING, at per complete cut				
2	5	Cuttings of ROADSIDE HAY MOWING, at per complete cut				
3	21.6	Miles of HAY MOWING, at per mile				
		TOTAL				

ITEM 1 LAWN TYPE MOWING PER COMPLETE CUTTING

Work under this item, if required under this contract, shall be performed in such areas and in such manner as specified for Item 1 mowing under "Scope of Work" and "Prosecution of Work" with the use of mowing equipment and tools that will satisfactorily cut all grass.

It may be necessary to remow portions or all of the area to eliminate all weeds or grass not cut to the specified height in order to complete each directed cutting in a manner satisfactory to the District Highway Engineer.
The Contractor shall not receive additional compensation for remowing necessary to produce a satisfactory cutting as directed by the District Highway Engineer.

Each cutting under this item shall be completed within 1 week from the date of beginning, which shall be Monday of a calendar week as directed, and any uncut and/or untrimmed areas or any unsatisfactory cut and/or trimmed areas remaining after 5:00 P. M. on Saturday of said week shall not be paid for.

In the event of such unsatisfactory areas remaining after 5:00 P. M. on Saturday, the Engineer shall estimate the number of hours which would be required to satisfactorily complete the work in said areas with use of the total normal complement of men and equipment assigned to the project by the Contractor. Two per cent of the unit bid price per complete cutting shall be deducted by the Department, for each hour so estimated by the Engineer to complete the work.

Rain or inclement weather shall offer no excuse for failure to complete each cutting within the time allowed except when in the opinion of the Engineer said rain or inclement weather is of such duration and intensity that work may not be performed, an extra day will be allowed for each day not worked as directed.

This work will be paid for at the contract unit price per complete cutting under Item 1 for Lawn Mowing, which price shall include full compensation for all labor, equipment, and necessary tools for this type of mowing, trimming and hand work required, transportation of equipment, tools and men to and from the site of work, operating expenses of equipment and all other incidentals necessary to satisfactorily complete the work.

ITEM 2 ROADSIDE HAY MOWING PER COMPLETE CUTTING

Work under this item shall be performed in such a manner as specified for Item 2 mowing under "Scope of Work" and "Prosecution of Work", with the use of mowing equipment and tools that will satisfactorily cut all grass.

Work under this Item shall also be performed in such areas as are specified for Item 1 mowing under "Scope of Work" on all projects which have excluded Item 1 from the proposal page.

It may be necessary to remow portions or all of the area to eliminate all weeds or grass not cut to the specified height in order to complete each directed cutting in a manner satisfactory to the District Highway Engineer, The Contractor shall not receive additional compensation for remowing necessary to produce a satisfactory complete cutting as directed by the District Highway Engineer.

Each cutting under this item shall be completed within 2 weeks from the date of its beginning, which shall be Monday of a calendar week as directed, and any uncut and/or untrimmed areas or unsatisfactory cut and/or trimmed areas remaining after 5:00 P. M. of the second Saturday shall not be paid for.

In the event of such unsatisfactory areas remaining after 5:00 P. M. on the second Saturday, the Engineer shall estimate the number of hours which would be required to satisfactorily complete the work in said areas with the use of the total normal complement of men and equipment assigned to the project by the Contractor. One per cent of the unit bid price per complete cutting, shall be deducted by the Department, for each hour so estimated by the Engineer to complete the work.

This work will be paid for at the contract unit price per complete cutting under Item 2 for Roadside Hay Mowing, which price shall include full compensation for all labor, equipment and necessary tools required for this type of mowing, trimming and hand work required, transportation of equipment, tools and men to and from site of the work, operating expenses of equipment and all other incidentals necessary to satisfactorily complete the work.

ITEM 3 HAY MOWING PER MILE

Work under this item shall be performed in such areas and in such a manner as specified for Item 3 Mowing under "Scope of Work" and "Prosecution of Work", with use of mowing equipment and tools that will satisfactorily cut all grass.

It may be necessary to remow portions or all of the area being worked in to eliminate all weeds or grass not cut to the specified height in order to complete this item in a manner satisfactory to the District Highway Engineer. The Contractor shall not receive additional compensation for re-cutting necessary to produce a satisfactory mowing within length being worked on.

The work will be paid for at contract unit price per mile of each completed mile regardless of the volume of work necessary in any particular mile. Each mile of mowing shall include both sides of the roadway. Work must be carried on in one direction and completed as work progresses, with both sides of the roadway kept uniform. Only the actual miles completed and satisfactorily mowed under Item 3 will be paid for.

The contract unit price per mile under Item 3 for Hay Mowing shall include full compensation for all labor, equipment and necessary tools required for this type of mowing, hand work required, transportation of equipment, tools and men to and from the site of the work, operating expenses of equipment and all other incidentals necessary to satisfactorily complete the work.

* * * * * * * * * *

PROPOSAL

FOR Mowing Grass along State Highways (Rtes. 28, 37, 139 & M)

COMMONWEALTH OF MASSACHUSETTS

LOCATION

The work referred to herein is in the Cities of Brockton and Quincy and Towns of Avon, Braintree, Holbrook, Randolph and Weymouth, Counties of Norfolk and Plymouth.

ROUTE		MILES
28	Avon, Brockton and Randolph	
	From the junction of Albion Street, Brockton, northerly to the junction of North Street, Randolph. Items 1, 2 and 3	4.0
	From Station 149+85, Randolph, at the railroad bridge, northerly to be excluding the Rte. 128 interchange, Randolph. Items 2 and 3	2.6
37	Braintree, Holbrook and Quincy	
	From the junction of West Street, Quincy, southerly to the junction of Royal Avenue, Holbrook, excluding the Route 128 interchange, Braintree. Items 2 and 3	6.2
	From Station 71+45 Holbrook, 700 feet north of Quincy Street southerly to the Brockton-Holbrook Line. Items 2 and 3	1.4
139	Holbrook and Weymouth	
	From the Abington-Weymouth Town Line, northwesterly to the junction of Weymouth Street, Holbrook. Items 1, 2 and 3	2.0
M	(East Main Street) Avon	
	From the Brockton Line northerly to the junction of Rte. 28. Items 2 and 3	0.6
M	(Old Route 128) Braintree and Weymouth	
	From the Weymouth-Hingham Town Line westerly to the junction of Route 37, Braintree, including Rte. 18 Intersection. Items 2 and 3	4.8
	Mowing Items 1, 2 and 3	
	Approximate Total Length	21.6

Example of Special conditions as would be noted on a proposal page under any given Route or Town description.

(Extending Item 2 beyond normal limits)
Note: On Route 128, Needham, from Station 123+00 at the Kendrick Street Overpass, northerly to Station 175+00 approximately 600 ft., north of the R. R. overpass, on the northbound barrel shall be mowed under Item 2 from the grass growth at the edge of the outer roadway to the layout line regardless of the presence of guard rail.

(Entire Proposal Item 1 & 2 areas treated with Growth Inhibitor MH-30. Item quantity would call for three Item 1 and two Item 2 cuts).

Note: All Item 1 and Item 2 grassed areas within the limits of this proposal have been or are to be treated with Maleic Hydrazide (MH-30) and as this chemical inhibits grass growth, it should be noted that the proposal calls for a reduced number of mowings.

Cuttings shall be made by the Contractor only when directed by the District Highway Engineer.

(Weekly Inspection Reports Accompanying Each Partial Pay Estimate)
COMMONWEALTH OF MASSACHUSETTS MNT. #111-R
DEPARTMENT OF PUBLIC WORKS

Form C-70

Contract No. ________ Week Ending ________________

Contractor ______________________ M-________-__________

To: Mr. ____________________

District Highway Engineer

Subject: Contract Mowing Inspection Report of Work for Payment.

Mowing was performed as described below along Route(s) ________________________

in ________________________________ a total length of ________________ miles

ITEM 1

Contractor began work ____________ 19__, Ended work ______________________ 19__.

Work (was) (was not) satisfactorily completed within the specified time.
______ hours are estimated to be required to complete the cutting in accordance with the provisions of the Contract.

Number of hours ____ x 2% = ____ per cent of cutting not completed, and to be subtracted from 100 to determine the amount for payment.

Per cent of cutting recommended for payment ____________.

ITEM 2

Contractor began work ____________ 19__, Ended work ______________________ 19__.

Work (was) (was not) satisfactorily completed within the specified time.
______ hours are estimated to be required to complete the cutting in accordance with the provisions of the Contract.

Number of hours ____ x 1% = ____ per cent of cutting not completed, and to be subtracted from 100 to determine the amount for payment.

Per cent of cutting recommended for payment ____________.

ITEM 3

Contractor began work ____________ 19__, Ended work ______________________ 19__.

Miles satisfactorily mowed ________________.

Contractor, (through his representative ____________________,) (was) (was not available to be) advised of the reasons for any and all reductions in payments covered by this report, the details of which are included in my diary.

Department Representative ________________
Signature

Copy to Contractor mailed ____________________________.
date

Copy to Maintenance Engineer (with pay estimate)

Bridge Deck Repair Techniques on the New Jersey Turnpike

ORRIN RILEY, Project Engineer; Howard, Needles, Tammen and Bergendoff

• NOWHERE have bridge deck repair problems been more vividly demonstrated, as to both variety and severity, than on the Passaic River Bridge of the New Jersey Turnpike.

Over 1 mi long and six lanes wide, the Passaic River Bridge stands in the Jersey meadows just outside of Newark. The $6\frac{1}{2}$-in. reinforced concrete deck on the bridge was placed during the winter of 1951. Since that time, the bridge has shown a steadily increasing volume of traffic now exceeding 80,000 vehicles per day, 19 percent of them trucks.

In 1957 spalling was noted throughout widely scattered areas on the bridge, although principally in the two outside lanes, which carry the brunt of the heavy truck traffic. This surface spalling increased considerably during the winter of 1957 and 1958 until, in March 1958, a deck slab failure occurred in one of the approach spans. Again, in June, several spans away from the first failure, a second slab broke through. These two failures, coupled with the realization that surface spalling was increasing beyond normal expectations, prompted the Turnpike Authority to undertake a comprehensive program to investigate and rehabilitate the concrete deck. The program may be divided into four broad categories: survey of condition, probable cause, methods of correction, and prevention.

SURVEY OF CONDITION

A continuing series of monthly inspections was begun. These inspections call for a foot-by-foot observation of the entire deck, both top and bottom, each month. Special report forms were prepared to plot the size and location of all cracks, spalls, and patches in every panel of every span in each lane. From these inspections, many data have been accumulated which have helped to evaluate the progressive stages of deterioration.

Generally there are three categories of distress: individual cracks, spalls, and grouped fractures called "checkerboard cracking."

Of the individual cracks, three different kinds may be defined:

1. Underside transverse cracks of random spacing and exhibiting efflorescence. These have been attributed to the effects of deadload deflections while the concrete was being placed. They are normally 3 to 6 ft long.
2. Full depth, full width, cracks probably caused by contraction, deflection, or both.
3. Shallow, top surface cracks in alligator pattern caused by rapid drying.

Spalling first appears as a small pock mark, generally not much larger than one's fist (Fig. 1). Usually, the appearance of a spall corresponds with the appearance of rust on the surface directly over a reinforcing bar. By tapping the area around these spalls with a small chipping hammer, a dull, hollow sound is often noted indicating that there is a much larger ring of unsound concrete surrounding the spall. Over a period of several weeks, and particularly during the winter, these spalls will grow rapidly in area and somewhat in depth until relatively large areas of reinforcement steel are exposed (Fig. 2). It is not uncommon for a spall to grow into a pothole 2 ft in diameter and 2 in. deep in one month's time. Normally, about 200 sq ft of new spalled areas are discovered each month, although twice, during wintertime, new spalls were created at the rate of over 1,200 sq ft per month.

Paper sponsored by Committee on Maintenance of Structures.

Figure 1. First stage spalling.

Checkerboard cracking is a phrase used to describe the phenomenon of underside cracks appearing with uniform spacing in both a longitudinal and transverse direction and showing efflorescence (Fig. 3). The passage of surface water through these checkerboard cracks during rain storms verifies that they are fractures through the slab. The spacing pattern conforms to the spacing of the lower mat of reinforcement steel. As many as twenty separate new areas of checkerboard cracking, varying in size from 1 to 60 sq ft, have been observed during one monthly inspection on the Passaic River Bridge. This condition is rarely encountered on other structures. Invariably, checkerboard cracking occurs beneath existing spalled areas or beneath patches made to repair old spalls.

Full slab failures have developed on four separate occasions. These have always occurred directly in the wheelpaths and in areas of checkerboard cracking. They are best described as holes about the size of a desk with chunks of broken concrete wedged between the two mats of reinforcement steel (Fig. 4).

PROBABLE CAUSE

It seems clear that the more serious conditions are merely the final phases in a sequence of progressive deterioration. Although no positive picture can be drawn of this sequence, the following theory seems the most likely.

Fine surface cracks admit water which during the winter months is usually salt water. Then, a combination of freeze-thaw action and the frequency of heavy axle loadings tends to widen and deepen these cracks. Eventually, the water or brine reaches the top reinforcing bars which begin to oxidize. The expansion of the steel during oxidation combined with further freeze-thaw cycles causes the cracked concrete over the bars to pop out, and a new spall is developed. The continued action of heavy traffic on the spalled edge abrades the area and increases the size of the spall. Then, impact loading on the reinforcement steel loosens the bond between the concrete and the reinforcement. The structural integrity of the deck slab has then been seriously weakened. Stress cracks begin to appear on the underside of the deck in a checkerboard pattern caused by the overstressing of the bottom layer of reinforcement steel. The deck is now supported only by the irregular shape of the fractured concrete pieces which are interlocked. Constant vibration and deflection wear the edges of these fractured pieces until one chunk falls through. Then the slab begins to break apart, and the roadway must be closed to traffic.

METHODS OF CORRECTION

After the various conditions had been documented, a three-point repair program was devised:

1. Completely replace all seriously damaged slabs and those that indicated incipient failure.

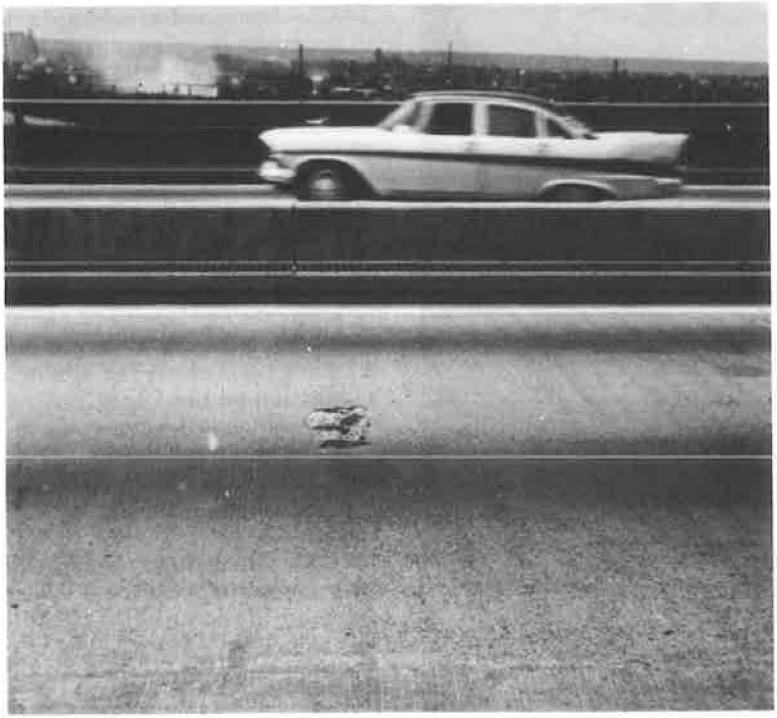


Figure 2. Second stage spalling.

Figure 3. Underside checkerboard cracking.

Figure 4. Slab failure.

2. Experiment with the most promising repair techniques for moderately damaged slabs.
3. Search for methods to reduce future spalling.

Deck Replacement

The criterion used for determining full deck replacement is the existence of underside checkerboard cracking. It is clear both from the history of slab failures and from plain common sense that a large area of fractured concrete beneath a deep pothole will not remain intact for long in a dynamically loaded deck. Therefore, each spring for the past three years all such slabs have been completely removed and replaced while keeping at least two lanes open to traffic in each direction.

The reconstruction procedure begins with a saw cut along the perimeter of the area to be replaced. Then the reinforcing bars are burned off, and the entire slab, including old reinforcement, is demolished and discarded. New forms are then set in the conventional manner except that the deck is increased in thickness to $7\frac{1}{2}$ in. by eliminating the haunch. To insure proper coverage of all reinforcing steel, special carrying bars are welded to studs at each mat elevation along the top of each stringer.

Concrete for the first reconstructed slabs was designed in accordance with the ACI recommendation (1). The design included air-entraining and water-reducing additives. Retarders or accelerators were used according to weather conditions and the requirements for reopening the lanes to traffic.

Figure 5. Nonshrinking concrete being placed over fresh conventional concrete.

On the whole this design proved quite satisfactory. However, after a year's time some of the replaced areas showed evidence of hairline cracks along the longitudinal joint between the new and old concrete. In addition there was a tendency for transverse cracks in adjacent old slabs to project across the new one. To offset this, a special metallic aggregate (Embeco), non-shrinking concrete topping was designed for use in later contracts. This new scheme called for concrete of conventional design to be placed in the lower $5\frac{1}{2}$ in. of the new slab. Then, while the lower portion was still in a plastic state, the top 2 in. was filled with the metallic aggregate concrete and finished in accordance with standard practices (Fig. 5). The specially prepared metallic aggregate used in the concrete mix has the unusual quality of expanding in volume during the setting and curing stages to form a void-filling, shrinkage-compensating system. The resulting concrete is strong, dense, waterproof, and elastic--precisely the characteristics sought for in bridge decks. The new concrete system has shown a remarkable resistance to the projection of cracks, and no defects have been noted to date. So far, about 20 percent of the old concrete deck has been replaced in this manner.

The cost of the two-layer concrete replacements is high, on the order of \$360 per cu yd, including demolition and new reinforcing steel but excluding traffic protection. However, all contracts have been awarded on an accelerated progress schedule and all prices reflect a great deal of overtime payment costs.

Figure 6. Excavated surface ready for nonshrinking concrete topping.

Surface Replacement

In areas where surface spalling has been extensive but where there is no checkerboard cracking, an intermediate technique called topping is often used. This technique requires the excavation of a large surface area, defined as something greater than 20 sq ft, down to a depth of 2 in. (Fig. 6). Great care must be taken in these larger topping areas to avoid damage to the reinforcing steel and to avoid overzealous use of cutting hammers which could break through the base concrete. With prudent care, however, this topping method can be much more economical than the making of many small patches because it requires less saw cutting, allows more productive use of mixing equipment, and permits the use of mechanized finishing equipment.

The concrete mix used in the topping method is the same in all respects as that used in the top 2 in. of the replacement slabs. But, because the nonshrinking topping concrete is being placed on an old concrete surface rather than against fresh, plastic concrete, the remaining old concrete that is exposed after excavation must be scrupulously cleaned and primed with a rich bond coat of metallic aggregate grout, generously applied.

About 20,000 sq ft of this type of topping construction have been used to date. So far only one minor edge failure at the corner of one slab has been noted. This probably resulted from improper compaction during placement.

Although nonshrinking metallic aggregate concrete is expensive, it affords many benefits that commend its use for bridge deck surfaces. The characteristics of high strength, high density, and crack resistance, make it an especially valuable material for repairing troublesome concrete.

Patching

In slabs that are generally sound and where spalls are too widely spaced to use topping, various patching methods have been used. Of the many important requirements in a good concrete patch, the most important of all is the removal of enough of the damaged concrete. Three or four times the area of the original spall must usually be removed.

The second most important requirement is saw-cutting of the patch edges. On the Passaic River Bridge, the rate of failure of feather-edged patches is from five to ten times higher than those made with saw cut edges (Fig. 7).

Figure 7. Pothole well prepared for patching.

As for patching materials, a variety of portland cement and epoxy resin products have been used. Of the portland cement products, the nonshrinking metallic aggregate concrete has been, by far, the most successful. After saw-cutting the perimeter and removing all the unsound concrete, the prepared holes are filled with water and left to soak overnight. The following day the holes are de-watered and a rich grout of metallic-aggregate mortar is brushed into the hole as a prime coat. Then a metallic-aggregate concrete with a 1-in. slump, similar to that used in the top of replacement areas, is placed and screeded.

The rate of failure of such patches is

approximately 3 percent. Nearly all concrete patch failures occur during the first few weeks and are attributed to the effect of live-load deflections acting on the deck and agitating the patch while it is still plastic. Such agitation inhibits the forming of a good bond. However, 4,600 sq ft of such patches remain intact on the deck today—many of them several years old.

Of the epoxy resin products, many various types and brands have been used over the past four years. Essentially, these have all been two-component epoxies with amine curing agents. The two parts are mixed with a coarse silica sand, in volumes varying from 1:1 to 1:5.

At first, experience with epoxy resin patches was rather unhappy, with one notable exception. Epoxy resin patches showed a rate of failure of 50 percent during the first year of traffic. In nearly all cases of such failure, peripheral cracks first appeared around the edge (Fig. 8). The patches then loosened and began to break apart under impact. This failure was caused by the inherent brittleness of many of the epoxy formulations used and by the different coefficients of expansion between epoxy and concrete. The development of peripheral cracks can be reduced by adding a flexibilizer to the epoxy formulation, but most of these flexibilizers tend to increase greatly the curing time and thus cancel one important benefit of using epoxies.

A highly successful exception to this experience is the coal-tar epoxy resin formulation (Guardcoat 140 resinous paving cement) which is a two-component epoxy with bitumen added. This material, when mixed at the ratio of 1:5 with 8-30 mesh silica sand has proved to be sufficiently flexible while maintaining a 2-hr curing time. To date, several hundred of these coal-tar epoxy resin patches have been installed, and so far, not a single failure has been noted. This emphasizes the importance of selecting the proper epoxy system for the job requirements.

As a result of experience, all Turnpike contracts now specify either nonshrinking metallic aggregate concrete, or coal-tar epoxy resin concrete, for all pothole patching. The criterion for selecting one or the other material is simply a matter of the time available. Where lanes can be closed down overnight or where concrete deck replacement is being made elsewhere within a lane, the portland cement material is used because the curing time is of no concern and the cost is slightly lower. But where lanes can be closed for only a few hours, coal-tar epoxy resin patches are used (Fig. 9).

The difference in cost was considerable at one time—ranging from \$20 to \$25 per sq ft for the epoxy resin patches down to \$13 per sq ft for the metallic aggregate patches, exclusive of traffic protection. The latest bid prices, however, have been more competitive, showing \$8.90 per sq ft for coal-tar epoxy resin patches and \$8.50 per sq ft for nonshrinking concrete patches.

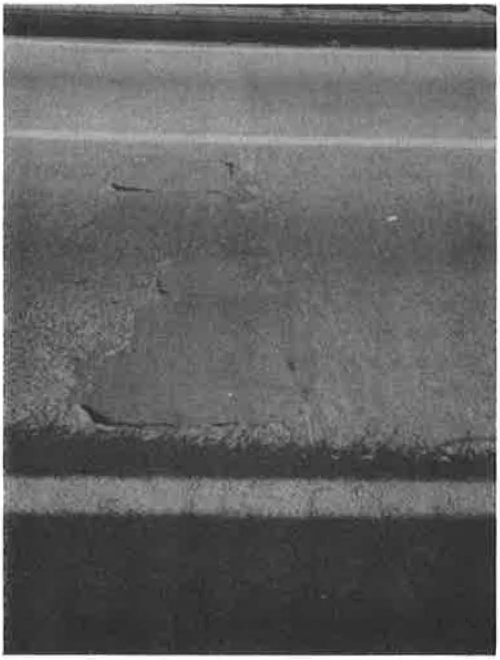


Figure 8. First stage of epoxy patch failure.

Surface Sealing

The final corrective method tried in the repair program was the use of thin waterproofing surface materials. The idea here, of course, was to seal both large and small cracks in an effort to cut off the first link in the chain of deterioration.

Silicone solutions were tried first. These were mixed with mineral spirits to

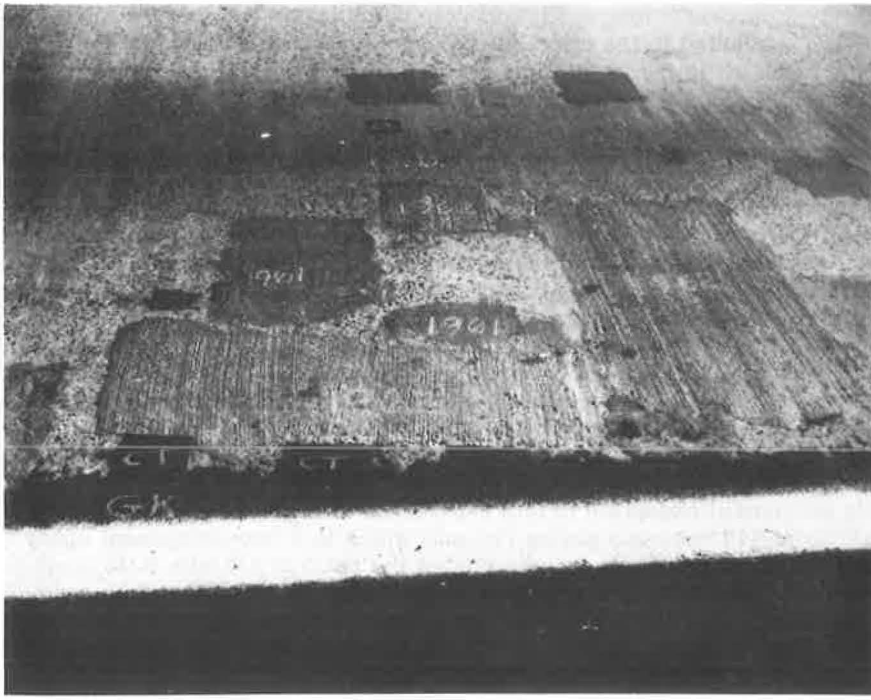


Figure 9. Group of successful patches made from nonshrinking concrete and coal-tar epoxy.

produce a solution containing 2 percent of silicone solids and this mixture was sprayed over the entire deck surface. This type of treatment had no measurable effect on the old concrete and spalling continued during the next two years at about the same rate as before. It is too early to tell what effect this treatment has had on the new replacement concrete.

Two years ago, a coal-tar epoxy resin roadway surfacing material was laid on 3,000 linear feet of one lane. This material was a two-component epoxy resin mixed and applied to a sand-blasted deck surface from a specially designed distributor truck at the rate of 3 lb per sq yd. While the epoxy was still wet, emery grit was sprinkled over the entire area to produce a tough, skid-resistant surface (Fig. 10).

This latter type of treatment has markedly reduced the rate of spalling in the old concrete. To date, an 80 percent reduction has been realized in the rate of new spalled areas created. As a further note, at least one-half the spalls that did occur under the resinous paving cement were

Figure 10. Coal-tar epoxy resin roadway surfacing.

confined to one nest in one slab, and that area of the slab soon cracked underneath. This failure was evidence of greatly weakened concrete, and indicated further that, in this one slab, the treatment was too late to expect much benefit. Curiously enough, the epoxy application also reduced substantially the number of old patch failures within the area—an unexpected bonus.

PREVENTION

Based on experiences with the Passaic River Bridge deck, revised standards for the Authority's new bridge deck design have been recommended. At the Lincoln Tunnel Interchange, now under construction, this design is being used. It features an 8-in. reinforced concrete slab constructed in the light of modern concrete technology, a coal-tar epoxy resin waterproofing sealer applied to the slab surface, and a $1\frac{1}{2}$- in. thick asbestos neoprene-asphalt wearing surface. This composite design of a structural concrete slab protected by an efficient sealer and a flexible wearing course is expected to produce a highly serviceable bridge deck that can resist weather attacks and sustain the tremendously increased traffic volumes on the New Jersey Turnpike.

ACKNOWLEDGMENTS

The author is indebted to the engineering staff of the New Jersey Turnpike Authority for their patience and cooperation, particularly to Ralph L. Fisher, Chief Engineer, for his helpful comments and suggestions and Arvind V. Kokatnur, Highway Design Engineer, who has supervised the work throughout this program. The author would also like to thank the many materials manufacturers and suppliers for their interest in and understanding of this work—especially C. V. Wittenwyler, Senior Chemist with the Shell Chemical Co., and Jack Weber of the Master Builders Co. for their technical advice.

REFERENCE

1. "Recommended Practice for Selecting Proportions for Concrete." Jour. Amer. Concrete Inst., ACI 613-54.

Discussion

M. SCHUPACK, of Schupack and Zollman, Stamford, Connecticut—The author is to be complimented on reporting the unfavorable as well as favorable remedial measures. It would be most helpful to the profession if more information on unsatisfactory incidences were made known.

Having worked as a designer on the Passaic River Bridge and having been in charge of checking the shop drawings for this bridge, the writer is familiar with the detail and design considerations of this structure. It appears that in all discussions of bridge deck problems, whether bridges of this size or for small bridges made up of stringers only, there is little discussion of the effect of secondary bendings introduced in the deck because of the bending of the deck in conforming to the deformation patterns of the total structure. These deformation patterns are based on the relative stiffness of the component parts of the structure and the types of connections made between various components. In any case it is generally very difficult to avoid some interaction stresses and these stresses should be given consideration in areas where members may be subjected to large stress if they interacted inadvertently. It appears to the writer that consideration should be given, particularly on the Passaic River Bridge, to the participation of the deck and the bending mode of the deck due to various loadings.

The major crossing of the Passaic River, which consists of 275-, 375- and 275-ft continuous spans, unlike the Hackensack River Bridge of the Jersey Turnpike, has details that tended to minimize completely longitudinal interaction of the bridge slab with the structural system. Sliding details are used on most stringers with a free joint every fourth floor beam. This was done to minimize the inadvertent participation

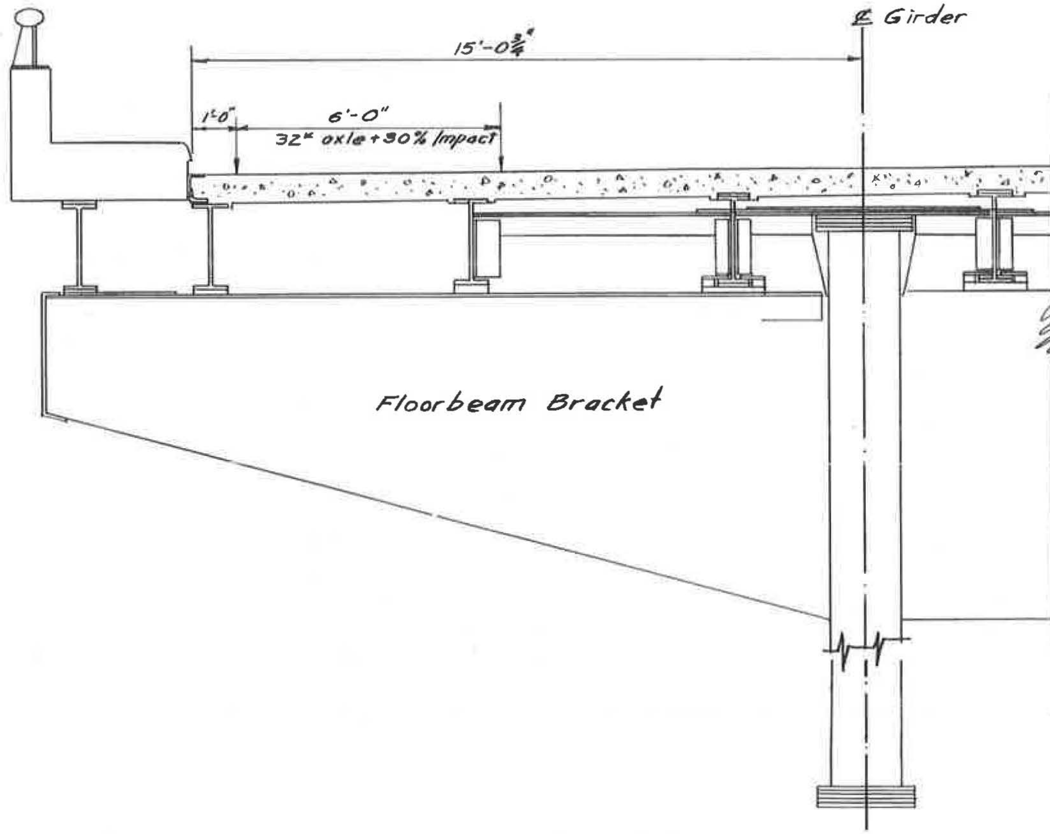


Figure 11. Cantilever bracket detail of floor beam.

stresses introduced into the tension splice of the floor beam to the floor beam bracket.

Figure 11 shows the cantilever bracket detail of the floor beam. The outside lane is fully carried by this cantilever. Because of the high traffic count of heavy trucks that run in the curb lane, it is very likely that the floor beam bracket receives almost the full design load many times a day. This is very unlike most structural elements used in bridge construction. In this condition and because of the details selected, it is not unlikely that the bridge slab tends to act as a partial tension flange for the floor beam. Axial tension stresses in the transverse direction would no doubt be very detrimental to the performance of this slab.

Approximate computations indicate a possible transverse axial tensile stress of about 70 psi in the slab due to a truck load in the curb lane. This stress was obtained assuming that the slab acted as a tension flange and the bottom flange of the bracket as a compression flange. With dual axles and a higher impact than 30 percent, this stress could be higher. Contributing to the problems of concrete placed under rather adverse construction conditions, over finishing, improper placement of reinforcing steel, and inadequate slab thickness, is the matter of the interaction of the slab in both longitudinal and transverse bending. The provision made in the steel details for minimizing the longitudinal interaction may have caused secondary problems not contemplated.

This condition indicates the need to give special consideration to the design of structural elements and assemblies whose predominant load is live load that is frequently applied. This condition is particularly critical where the impact may exceed the maximum of 30 percent usually specified.

ORRIN RILEY, Closure—Mr. Schupack has presented a thorough and interesting analysis of one of the primary factors that may have contributed to the cause of the initial cracking in the Passaic River Bridge deck. Because his discussion is concerned primarily with considerations for preventing recurrence, rather than maintenance and repair, it is beyond the scope of the original paper. Nevertheless, he has made a valuable contribution to the literature of improved bridge deck performance.

Galvanizing Reduces Bridge Rail and Guardrail Maintenance in Michigan

S. M. CARDONE, Senior District Engineer, Michigan State Highway Department

• THE MICHIGAN State Highway Department operates and maintains a trunkline system of some 9,200 centerline miles. The roads vary from 2-lane 20-ft widths to 4, 6 and 8 lanes in certain urban areas of heavy traffic concentration. Two thousand miles of this system, 50 percent completed, is in freeways consisting of Interstate and arterial highways. Most of the freeway mileage is in the form of limited-access highways, which consist of two widely divided, 24- or 36-ft paved strips, each carrying one-way traffic only.

At present this system includes about 2,500 bridges (3,000 when current construction is completed in a few years), all of which conform to the standards of the American Association of State Highway Officials. According to Michigan classification, a "bridge" starts at a 20-ft span.

BRIDGE RAILINGS

The bridge railings, like the bridges, vary widely in concept, structure, and basic construction materials. Stone, precast concrete, ornamental ironwork in numerous configurations, and aluminum, are serving as the protective bridge railings. Most iron bridge railing has in the past been specified as black steel which, having the necessary inherent factors of strength and impact resistance, has served well. Approximately 53 miles of railing of this type are currently being utilized.

Protection of this bridge railing from weather corrosion has involved substantial maintenance. Speaking generally, the steel railings called for repainting every three years. They were not necessarily painted every three years, but this was the cycle which experience indicated as most desirable.

Sandblast cleaning was impractical, considering the openwork design of the railings. Therefore, the steel was cleaned with power-operated and hand brushes and scrapers. Spot priming, and even complete repriming with a top quality red lead primer, was followed by a surface coat of aluminum paint.

Prior to World War II the total cost for repainting these railings was about $1 per linear foot. After the war, as wages and material costs rose sharply, maintenance costs for bridge rail repainting began to run as high as $3 and $4 per foot. One pilot job of railing painting performed in October 1962, primarily for reporting in this paper, and typical in respect to condition of protective coating, cost $4.45 per linear foot. Some 80 percent of the total cost, of course, was in the cleaning and preparation of the surface before painting. For 50 miles (264,000 ft) of steel bridge rail at $4.00 per foot, $1,056,000 would be needed every three years (or $352,000 a year) for painting bridge rail.

In 1956 one section of bridge rail was cleaned and hot-dip galvanized to study its corrosion endurance as compared to that of painting. After two years of exposure no deterioration was visible, although paint of similar age would have begun to show considerable signs of failure. Accordingly, the rails of the entire bridge were galvanized. A steel fabricator, under contract, removed the railings, trucked them to a hot-dip galvanizer where they were cleaned and galvanized to a 5-mil or 3-oz specification, returned them to the bridge, and reinstalled them. The contractor also erected temporary pipe rails as a safety precaution while the permanent railings were being galvanized.

Paper sponsored by Committee on Maintenance of Structures.

The cost ($8.50 per foot) was not economical, but analysis of the operation showed many steps which could be taken to lower the costs approximately one-half, as discussed later.

This bridge was inspected periodically during three years to provide performance data (actually 6 years of exposure in the case of the single section of railing that had been galvanized in 1956). Observations throughout this period, when paint would have failed, gave convincing evidence that galvanizing offered excellent service life. No surface deterioration was observed during this period. Consequently, in 1961 galvanization of bridge railings was adopted as a basic maintenance program throughout the State.

Current costs come to $2.25 a foot for the stripping and galvanizing with approximately $3.00 a foot to be added for the operations of removal, trucking, return, and re-erection. Regular highway department crews and trucks are used for this work. The temporary safety railings erected during the work consist of old cable guardrail supporting snow fencing. The work is done largely in the colder months of the year when structure maintenance is at a minimum and personnel are most readily available. Safety was naturally a principal consideration during the period when temporary railings are in place, but experience indicates that the incidence of bridge impacts by cars is actually lower in winter months than in summer. Apparently, greater driver care and attention is enough to offset the often more dangerous driving conditions.

Alternatives to provide satisfactory protective coatings have been tested. One bridge was metallized with aluminum; but the cost was high, minute rust pock marks developed at an early date, and some peeling of the coating is taking place. Epoxy resin paints were also tested, but spot failures began to appear after only one winter's exposure.

GUARDRAIL

A change in organizational structure of the Michigan State Highway Department brought bridge maintenance and highway maintenance together in one organization in 1959, at which time the analysis applied to protecting bridge rail from corrosion was extended to include highway guardrail. The department used black steel plate guardrail approximately two years before changing to galvanized beam. In these two years 25 miles of guardrail were constructed. Of this length, approximately 75 percent has been recoated with hot-dip galvanizing. Currently all new guardrail is of steel, hot-dip galvanized before fabrication.

Originally, maintenance of guardrail consisted of cleaning with hand tools such as wire brushes and scrapers. This was followed by one coat of good red lead primer and a top coat of white paint. Repainting of guardrail was required on a shorter cycle than that required for bridge rail. Owing to the concentrated exposures to salt in snow and slush and to the physical impact of snow removal equipment, normally guardrails needed repainting every one to two years.

The cost of painting guardrail was most recently $0.40 a foot. The cost of galvanizing is about $0.30 a foot, with the cost of removing, trucking and replacing adding another $0.20 a foot. In at least one district, galvanized guardrail is installed on a rotating basis to replace the guardrail being removed for galvanizing, thereby avoiding double handling and the need for temporary protection.

CONCLUSIONS

As a result of experience to date, the Michigan State Highway Department now purchases no steel bridge rail for maintenance purposes or highway guardrail which is not hot-dip galvanized. The latest design of bridge railing uses a one-tube galvanized steel rail over a 22-in. parapet wall, making an overall railing height of 36 in. Inspections of galvanized guardrail over a 4-year period have shown no surface deterioration, as compared to the 1- to 2-year life expectancy of paint systems. In the future, it is planned to protect all highway guardrail and bridge rail by regalvanizing when it becomes necessary. The anticipated service life is not known precisely, but 8 to 10 years is expected for highway guardrail and 12 to 15 years for bridge rail. Perhaps galvanizing may have a part in other areas of the department's operation. For example, a highway bridge in which a 34-ft span is protected by hot-dip galvanizing is currently under exposure test. If exposure results warrant, it is hoped to adopt this form of protection for large structural bridge members.

Another area of promise for large structural steel beams may lie in the field of metallizing with zinc.

Rising prices have brought painting and hot-dip galvanizing to roughly comparable cost ranges for initial application. Because the service life expectancy of galvanizing is in the nature of four or five times that of paints, there is little question as to the desirability and the substantial savings to be expected from the decision to drop painting in favor of galvanizing in the maintenance program.

Discussion

JOHN R. DAESEN, Director, The Galvanizing Institute, Park Ridge, Ill.—Highway engineers will find Mr. Cardone's report of the satisfactory and economical use of hot-dip galvanizing for protection of steel bridge rail and guardrail of value. This discussion is meant to provide additional information regarding the life expectancy of the coating and the most feasible and economical methods of maintenance.

The life will depend largely, but not entirely, on the weight of coating. Most of the bridge rail and guardrail hot-dip galvanized after fabrication is specified to carry a minimum average coating of 2 oz per square foot of surface (equivalent to 3.4 mils thickness) in accordance with ASTM Specification A-123. With such a coating in a moderately industrial atmosphere a life of at least 20 years may be expected before the first appearance of rust of the base, rather than the 8- to 15-year life mentioned by the author. Complete failure of the coating might be expected no sooner than about twice the time to first rust. This presumes no conditions of continuous immersion in water.

It is noted that Mr. Cardone's department specifies guardrail of steel hot-dip galvanized before fabrication. Aside from differences in chemical composition and structure of this type of coating, as contrasted with the galvanized after type of coating on which the Michigan Highway Department experienced such fine results, the galvanized before fabrication material is often furnished to meet ASTM Specification A-93 with a much lighter weight of coating. The highest weight class of A-93, 2.75 oz (pot yield) per square foot of sheet, not surface, has a minimum weight, determined by averaging two sides of a 5.06-sq in. test sample, that is only 1 oz per square foot of surface, or one-half that of the weight required under ASTM Specification A-123.

Assuming that this heaviest class of coating under A-93 is used, the life to be expected from this galvanized before fabrication material is one-half that of the galvanized after fabrication, meeting ASTM Specification A-123. As Mr. Cardone's figures are in line for cost of guardrail galvanized after fabrication, highway officials will find it profitable to examine this difference closely.

The weight difference is not the only important difference, however. Some recently produced 0.109-in. (12-gage) steel guardrail, galvanized before fabrication and exposed only a few months, although still exhibiting the spangles and lustre, have many fine pits in the surface. The source of this unsoundness of surface is found to be oxide inclusions in the coating, originating from the use of a higher amount of aluminum (0.05 percent or more) in the galvanizing bath than is used or permitted in ASTM Specification A-123.

The structure of the coating on ¼-in. steel galvanized after fabrication discloses that the zinc-iron alloy layers that bond the outer zinc layer to the steel base are very prominent in the galvanized after coating, but are greatly reduced, and in some areas almost entirely lacking, in the galvanized before product. This is the principal difference in the structures of the two products.

The practical elimination of the intermediate alloy layer promotes ductility to permit severe forming, such as bending or seaming, and most of the thin gage strip galvanized in continuous coils is made using an increased amount of aluminum to practically inhibit formation of the alloy layer. Such ductility is not needed, of course, in guardrail, where material with substantial alloy layers with a coating 3.4 mils thick has completely adequate ductility to permit moderate bends or straightening out after distortion by impact.

The alloy layers of galvanized after fabrication coatings tend to maintain uniformity of thickness of coating, because their growth by diffusion in the bath is regular and controlled by time and temperature of immersion in the galvanizing kettle. The bulk of the coating on the material galvanized before fabrication, which contains enough aluminum to greatly restrict formation of the alloy layer, is controlled largely by drainage of molten zinc from the work. It is for this reason that coatings containing enough aluminum to greatly restrict alloy formation (0.05 to 0.15 percent) are seldom produced with weights of coating in excess of 1 oz per square foot of surface.

The coating on 0.036-in. (20-in. gage) steel sheets galvanized by the older process, in which aluminum in the molten bath was kept at a value of 0.01 percent or less, is broken up by more massive inclusions, in this case of flux, resulting from entrapment of the molten flux particles on the sheet. At the high speed of travel of these sheets through the galvanizing bath, there was much less time for this material to wash off, as compared with the longer time of immersion in the hot-dip galvanizing of structurals, or guardrail or bridge rail galvanized after fabrication. The fact that many of these flux inclusions open up to the surface of the sheet explains why these sheets were darkly stained although never exposed out-of-doors.

The thin discontinous alloy layer on the 0.109-in. gage guardrail, compared with that of the 0.036-in. and ¼-in. gage material, indicates an aluminum content of about 0.05 percent in the galvanized before fabrication guardrail, a value which neither completely inhibits the formation of alloy nor allows it to grow at the normal rate. The rapid destruction of the flux, with attendant problems of oxide inclusions, is the reason why aluminum contents of this magnitude are prohibited in ASTM Specification A-123 (aluminum 0.01 percent maximum).

These defects have their effect in reducing the life of the coating and causing unattractive darkening of the surface. The photomicrographic evidence is shown, not to prove relative quality, since good and less desirable structures may be found in all manufactured materials, but to explain differences that occur on test and in use that otherwise might be erroneously charged to mere spread of values, inevitable in production operations. It will be seen that such structural differences are related to differences in manufacturing practices used in producing the two types of materials.

Although Mr. Cardone indicates that the Michigan Highway Department anticipates regalvanizing the bridge and guardrail when necessary, it will probably be found, that after the coating has gradually weathered away so that the corrosion product turns tan or brown, indicating that the zinc-iron alloy layers are gradually dissolving, it will be in order to paint the rail.

The objection to high cost of painting will not apply here, because there will have been no pitting of the steel and preparation will consist only of hand brushing instead of expensive sand blasting. A treatment by phosphate coating or a coat of wash primer is required before applying the paint unless the paint is a zinc-dust paint, a cement-base paint, or the new calcium-ortho-plumbate type.

Such pretreatment is even more necessary when galvanized structures are painted (for reasons of visibility or appearance) when they are first installed. The adherence of paint coats is affected by differences in the type of galvanizing (low or high aluminum content) and the type of steel base. White specks of zinc oxide are sometimes formed with the gray basic zinc carbonate in normal weathering of zinc or sound galvanizing. Failure of the paint coat to adhere can often be laid to improper selection or application of preparatory treatment, as the metal base remains normal.

The roughness of galvanized coatings is sometimes the result of differences in the steel base, although rough galvanizing can occur on any base improperly handled. Pits due to rough galvanizing may "fester," lifting the paint coating and causing failure. Dark staining on the weathered galvanized surface shows the effect of these pits.

Paint producers have given increased attention to providing for differences in the type of galvanized coatings. They can provide foolproof systems for use on coatings of any type of manufacture, but the type of galvanizing must be considered and the coating must be sound.

Rehabilitation of Two Concrete Arch Bridges

VERE P. MAUN, Principal Civil Engineer (Design), New York State Department of Public Works

This paper has been prepared as an extension of the paper on "Examples of Repairs to Concrete in Bridges" (HRB Bull. 323).

During 1962 the extensive repairs being made to two long concrete arch bridges at Troy, and Glens Falls, N.Y., were completed. This paper discusses the extent of the damage encountered, the methods used to repair and protect the concrete, and the unit costs of the various contract items.

• AT THE 1962 HRB Annual Meeting, the author gave a paper discussing methods being used to repair and protect concrete damaged by de-icing chemicals. Now the work is sufficiently finished to permit showing before and after pictures and adding to the previous discussions.

This paper discusses the repair and protection of the concrete in two Hudson River concrete arch bridges, made necessary by the use of de-icing chemicals. Only time will tell as to the effectiveness and permanence of the work done on these two structures.

The first bridge repaired was the 112th Street Bridge over the Hudson River between Troy and Cohoes, N.Y. This bridge was built by the State of New York about 1920. It consists of several long, concrete arch spans, open spandrel type with solid concrete arch barrels. The span over the navigable channel consists of a double-leaf trunnion-type bascule bridge with a clear span of about 200 ft. It carries about 10,000 cars per day, and salt is used freely on it for snow removal.

The sidewalk railings on the concrete spans are of the heavy, open parapet-type and the bridge has several ornamental pylons on it plus four concrete houses about 20 ft high above the sidewalks at the ends of the bascule spans. The roadway of the arches rests on a 12-in. concrete slab supported by transverse spandrel walls. This bridge at one time carried two streetcar tracks (in addition to highway traffic) until the streetcar lines were abandoned. The rails were set in a fill concrete carried on the 12-in. arch slab. The fill concrete was covered with an asphalt wearing surface.

When the bridge was closely examined before beginning plans for repair work, the following conditions were evident:

1. The concrete fill under the roadway was breaking up.
2. Curbs were badly disintegrated.
3. Sidewalks had large areas of spalled surfaces.
4. Railings and pylons had many disintegrated places on them (Fig. 1).
5. Retaining walls and vertical spandrel walls were spalled and unsightly (Fig. 2).
6. The exposed bottom surface of the arch barrels had holes and other disintegrated areas showing.
7. Many of the steel plates on the bascule span needed replacing, having been eaten away by corrosion.
8. The timber block floor on the bascule span was disintegrated and broken up, plus being slippery in wet weather, causing many accidents.

Paper sponsored by Committee on Effect of Ice Control.

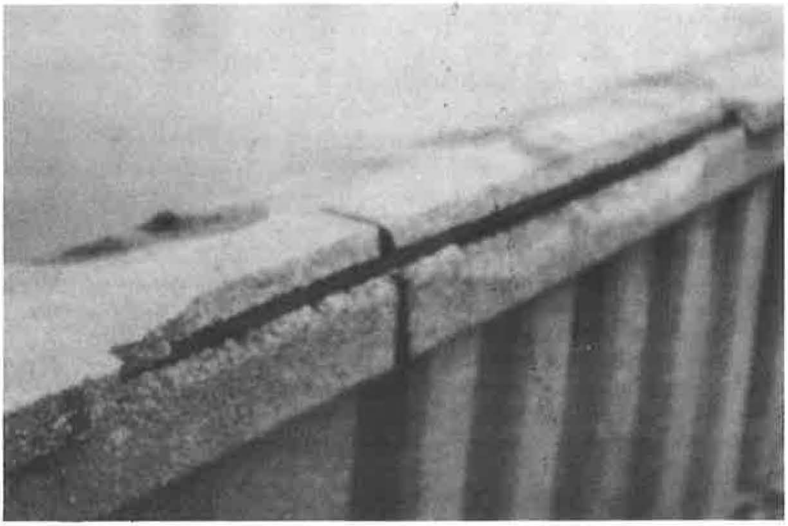


Figure 1. Sidewalk railing before repair, 112th St. Bridge, Troy.

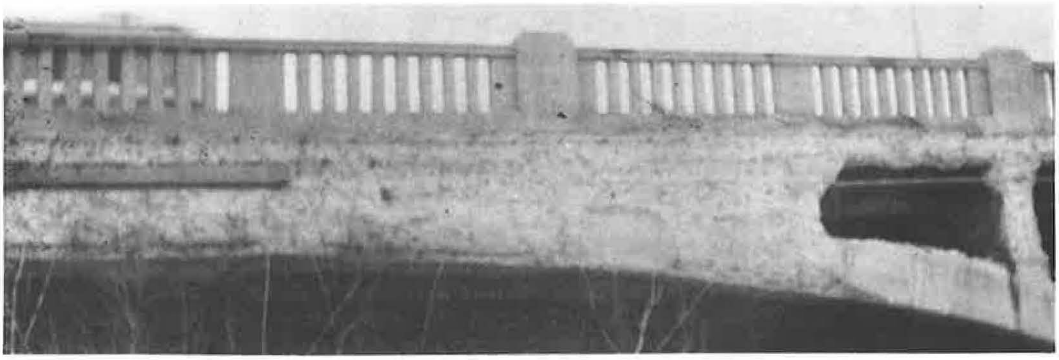


Figure 2. Spalled arch ribs before repair, 112th St. Bridge, Troy.

9. Corrosion had weakened the steel railing posts of the bascule span.
10. The steel in the bascule span needed cleaning and painting.
11. The electrical system, including motors of the bascule span, needed overhauling. One leaf of the bascule could not be opened.

It was estimated that it would cost over $300,000 to rehabilitate the bridge, and a contract was let to do the work. When work started, about 12 in. of fill concrete, wearing surfaces, and old steel ties were removed down to the arch slabs. A new air-entrained concrete fill was poured with the proper crown. It was then covered with silicone and then fiberglass coating. Over the fiberglass coating was placed 2½ in. of asphaltic concrete as a wearing surface. The curbs were cut back with saws and jack hammers and replaced by granite curbs. Damaged areas were sawed and removed by jack hammers to a depth of about 2 in. and repaved with new concrete rich with air entrainment. This new concrete and all old sidewalk areas were covered with epoxy bonding compound and then sprinkled heavily with silica sand. The epoxy was to protect sidewalk surfaces from salt damage and the sand was added to give pedestrians good traction, as epoxy is slippery without sanded surfaces (Fig. 3).

Spalled places on railings, pylons, parapets, arch barrels, and other places had

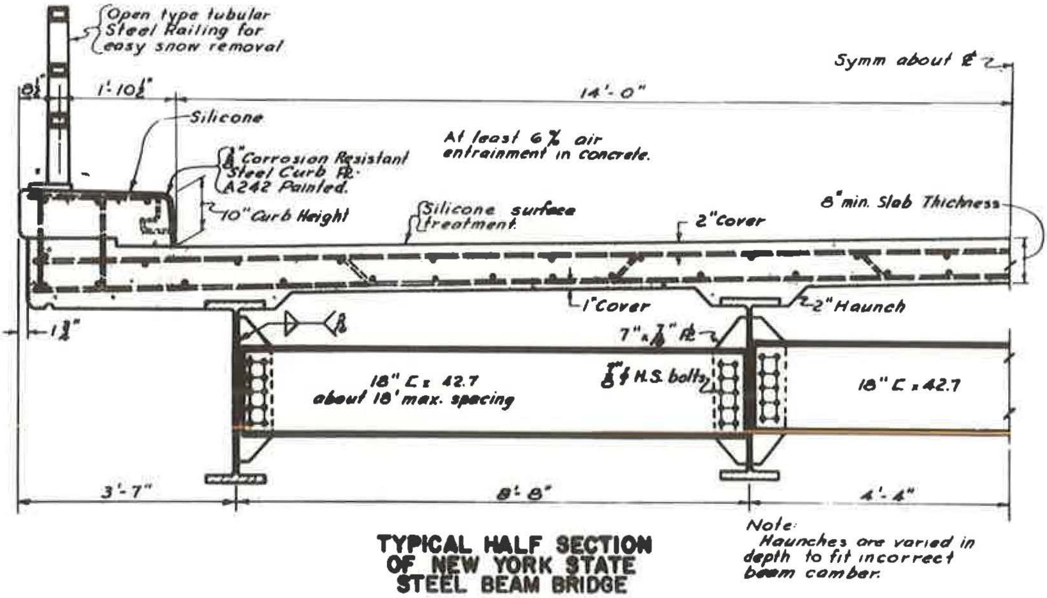


Figure 3. Method used to protect steel I-beam bridge decks from salt damage.

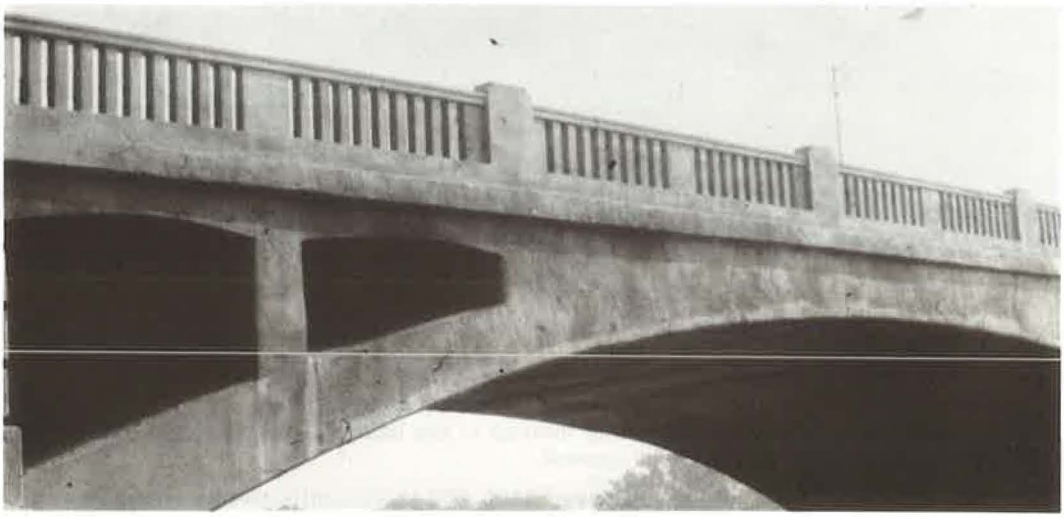


Figure 4. Railing after repair with gunite and waterproofing paint, 112th St. Bridge, Troy.

bad concrete removed, then were coated with an epoxy bonding compound before applying gunite (Fig. 4). The proper outlines were restored by use of shooting strips, rubber trowels, and other means known to guniting operators. Finally, all concrete surfaces were sprayed with two coats of a waterproofing bakelite paint (Penetryn). The second coat had a color to it which made all concrete surfaces have the same color and which greatly dressed up the concrete in appearance (Fig. 5).

Figure 5. Arch rib after repair with gunite; waterproofing paint not yet on, 112th St. Bridge, Troy.

Figure 6. 112th St. Bridge, Troy, after repair; new roadway slab and steel grating on bascule span, new granite curbs, sidewalks repaired and protected by epoxy bonding compound covered with silica sand for traction, all concrete repaired and waterproofed.

The steel in the bascule span was repaired, cleaned, and painted. A new 5-in. steel grating floor was placed on the roadway using steel channels for curbs. A 1-in. grating floor was placed on the sidewalk and this grating was filled with lightweight concrete. New steel railings were put on the bascule span (Fig. 6). All machinery and electric motors were cleaned and rehabilitated. New light standards and lights were put on the bridge and new electric gates were installed for halting bridge traffic when the bascule was open.

Traffic was maintained during most of the work, but it was necessary to close the

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SHEET 1 OF 2
ENGINEER'S ESTIMATE

- 4 - Copies

COUNTY: Albany-Rensselaer
Troy(112th St.)-Cohoes Bridge
Over Hudson River

Repair of 4 R.C.Arches; Type 2 @ 121'-6", 2 @168';
Type Ctr.Span-D.L. Bascule-224' Long & Approaches
Thickness

HIGHWAY No.
City of Troy Miles 0.09 St'd Sheets
City of Cohoes Miles 0.13 St'd Sheets

Date
Total Length= 1144 ft.= 0.22 Miles
Bridge Width 41'-0" Pav. 27'-0" R'way

No.	ITEM	Unit	QUANTITY	PRICE	AMOUNT
1	Clearing and Grubbing	L.S.	100%	600.00	600.00
15-2-A	Portland Cement - Type 2A	Bbl.	638.5	4.50	2,873.25
18X	Class 1A Concrete for Structures	C.Y.	39.7	40.00	1,588.00
18XL	Light Weight Conc. Floor Filler	C.Y.	22.2	55.00	1,221.00
25F	Steel Fabric Reinforcement	S.Y.	1,995	0.60	1,197.00
25FS	Steel Fabric Reinforcement	S.F.	10,252	0.48	4,920.96
28RR	Bar Reinf. for Structures	Lbs.	5,214	0.20	1,042.80
29X	Structural Steel	Lbs.	112,920	0.40	45,168.00
30BCX	Open Steel Floor (Roadway)	S.F.	5,629.5	6.27	35,296.97
30BCY	Open Steel Floor (Sidewalk)	S.F.	2,272.5	2.70	6,135.75
30S	Miscellaneous Metals	Lbs.	3,248.5	1.50	4,872.75
32X	Cable Guide Railing	L.F.	422.5	3.15	1,330.88
37A	Metal Railing	L.F.	490	15.00	7,350.00
47BR	Cement Concrete Pavement	C.Y.	318	28.00	8,904.00
51M	Asphalt Concrete	Ton	454.6	11.00	5,000.60
51TCM	Stab.Grav.Mix Bit.Treat.Shoulder	C.Y.	11.5	10.00	115.00
70B	Bit.Mat."A" Emul., Grade C	Gal.	48	0.30	14.40
70M	Bit.Mat."A" Emulsion	Gal.	164	0.25	41.00
76Y	Main. & Prot. of Traffic-Reg. A	L.S.	100%	12000.00	12,000.00
81X	Removing Existing Bridge Deck Arch Span	L.F.	665	17.00	11,305.00
81Y	Removing Existing Bridge Deck - Bascule Span	L.F.	215	35.00	7,525.00
81Z	Removal of Approach Pavement	S.Y.	157	2.50	392.50
95S	Stone Curb - Granite	L.F.	1,331.5	4.26	5,672.19
111S	Pneumatically Projected Conc.	Bags	4,026	18.00	72,468.00
121A	Topsoil Furnished and Placed	C.Y.	87.6	10.00	876.00
123S	Seeding on Special Areas	Acre	0.08	1500.00	120.00
211	Clean & Paint Steel Basc. Span	L.S.	100%	12000.00	12,000.00
212	Orange Primer Paint M20A	Gal.	70	6.50	455.00
213	Gray Paint M23A	Gal.	240	4.00	960.00
214	Gray-Green Paint M24A	Gal.	270	4.00	1,080.00
215	Miscellaneous Painting	Gal.	23	20.00	460.00
219	Repairing Structural Concrete	C.Y.	10.8	150.00	1,620.00
220	Revising Concrete Curb	L.F.	1,331.5	3.50	4,660.25
221	Balancing Bascule Spans	L.S.	100%	2170.00	2,170.00
222	Pig Iron Balance Blocks	Ton	0	250.00	0.00
305	Epoxy Bonding Compound	Gal.	299.7	25.00	7,492.50
306A	Concrete Waterproofing Penetrating Coat	Gal.	1,810	7.80	14,118.00

State pays ______ % $ ______
______ pays ______ % $ ______
______ pays ______ % $ ______

Cost of work ______ $ ______
Contingencies ______
Eng.. adv., etc. ______
Total cost ______ $ ______

DATE:
REVISED BY:
CHECKED BY:
COMPILED BY:

Figure 7. List of contract items.

SHEET 2 OF 2

ENGINEER'S ESTIMATE

Form C-114H

STATE OF NEW YORK

DEPARTMENT OF PUBLIC WORKS

DIVISION OF CONSTRUCTION—HIGHWAYS

Copies

COUNTY Troy(112th St.)-Cohoes Bridge Over Hudson River

Date

Length= ft.= Miles

Type HIGHWAY No.

Width Pav. R'way

Type Town of Miles St'd Sheets

Thickness Town of Miles St'd Sheets

DATE: REVISED BY: CHECKED BY: COMPILED BY:

No.	ITEM	Unit	QUANTITY	PRICE	AMOUNT		
306B	Concrete Waterproofing Finish Coat	Gal.	1,678.5	12.50	20	981	25
307	Cleaning Concrete Surfaces	L.S.	100%	5000.00	5	000	00
352X	Rubber Joint Material	Gal.	48.75	30.00	1	462	50
401	Overhaul & Clean Electrical Equipment	L.S.	100%	3000.00	3	000	00
402	Electrical Work	L.S.	100%	32900.00	32	900	00
403	Clean & Lubricate Mechanical Equipment	L.S.	100%	890.00		890	00
405B	Fiber Glass Reinforced Resin Coating	S.F.	18,342	1.30	23	844	60
	FINAL ESTIMATE		TOTAL		371	125	15

State pays % $

pays % $

pays % $

Cost of work	$ 371,125	15
Contingencies		
Eng., adv., etc.		
Total cost	$	

Figure 7. Continued.

bridge for a period of about nine weeks while working on the bascule span. One leaf of the bascule span was kept in operation for navigation purposes at all times during the navigation season. Total cost of the work on this structure was over $375, 000;

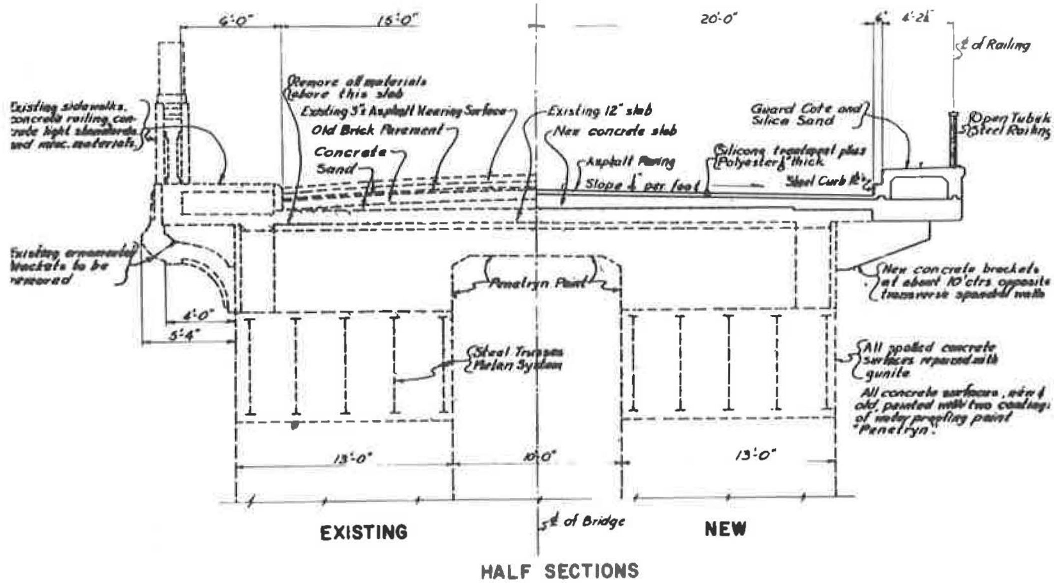

Figure 8. Rehabilitation of Glens Falls Bridge: left half, how bridge was built; right half, how bridge was repaired and widened.

Figure 9. Glens Falls Bridge concrete arches, deteriorated concrete being removed.

about $75,000 of this was spent on electrical and machinery work. A complete list of contract items with the unit bid price actually paid for each item is shown in Figure 7.

The Glens Falls bridge was built in 1915 by the Cities of Glens Falls and South Glens Falls. It was taken over by the State of New York about five years ago as part of the State highway system.

It consists of several concrete fixed arches, open spandrel type with all piers and abutments resting on exposed rock. It is the third bridge at this site, the first being built about 1800. There is a rocky gorge under the bridge and a power dam above the bridge. Water is often going over the dam, causing a mist which often covers the bridge. In freezing weather, the mist causes the sidewalks and roadways to be covered with ice, thus requiring frequent salting to keep the bridge passable for cars and pedestrians. The river water is also badly polluted with sulfides from the paper mills

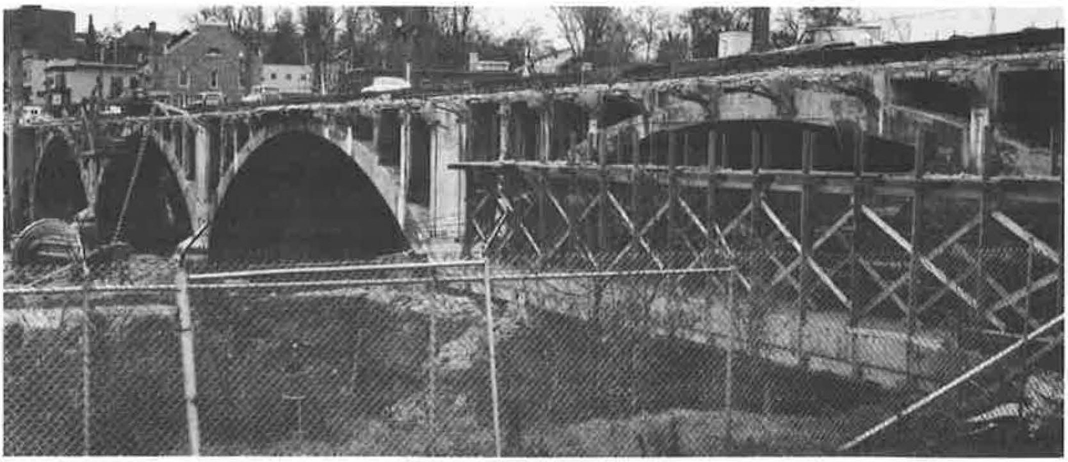

Figure 10. Glens Falls Bridge concrete arches after railings and wearing surfaces have been removed preparatory to repair and widening.

Figure 11. West side of Glens Falls Bridge after being repaired and waterproofed.

above the bridge; the sulfides appear to have also acted to deteriorate the concrete along with the chlorides applied by salting.

All but two of the arches are of the ribbed type and the Melan system of reinforcing and erection was used. Transverse spandrel walls transfer down to the arches the roadway loads and the weight of the 12-in. slab on which the roadway was built. The other two arches have solid barrels and are filled.

In the article mentioned at the beginning of this paper, this bridge was discussed to some extent; now most of the rehabilitation has been completed (Figs. 8 through 11).

The following materials and methods are being used to restore deteriorated concrete and to protect concrete from the corrosive effects of de-icing chemicals. Gunite is the most useful tool for patching surfaces and holes formed by decayed concrete. Where concrete can be formed, air-entrained concrete has been used. Epoxies are used to bond new concrete to old concrete and also as a protective coating over concrete. For curbs, either granite is used or concrete curbs are protected by corrosion-resistant steel plates. Concrete surfaces are painted with waterproofing materials such as Penetryn. The Department is trying to drain joints so that salt water will not fall on concrete or steel. By use of continuous and cantilever spans in steel work, most of the joints over piers can be eliminated. The Department is trying to improve the concrete and the curing of the concrete by tightening up concrete specifications both in the line of materials and in the manipulations of the concrete while being mixed and placed.

Salvaging Rigid Pavement in Georgia

W. F. ABERCROMBIE, State Highway Materials Engineer, State Highway Department of Georgia

This paper describes the methods of salvaging rigid pavements in Georgia. The use of the pavement breakers for seating rocking slabs and removal of extremely broken-up pavement and replacing with premixed soil-bound macadam portland cement-stabilized base course mixture are described. The methods of salvaging old pavements have been used on many miles and in several contracts. The results of these methods as found by inspection after some service are also included.

THIS PAPER presents an alternate method to that of seating old rigid pavements with heavy pneumatic-tired rollers. The alternative method is being used by the State of Georgia and is presented for comparative purposes to the use of the heavy pneumatic-tired rollers.

The problem of rehabilitating old pavements is one that all Highway Departments face sooner or later. It is a problem that requires intricate study of each project as to its condition. Further, it requires the determination of the best methods and design to give a long and satisfactory life in accommodating the anticipated traffic in the future as well as accomplishing the rehabilitation in the most economical manner possible.

Georgia has many old rigid pavements that through the last few years have become so broken and rough as to require some type of rehabilitation (Fig. 1). These old pavements are faulted at the joints, with considerable breakage of the slabs along the joints and cracks, rocking panels or slabs and warped panels. In many instances, study of the projects for the purpose, of choosing the type of rehabilitation showed cavities existed under the slabs of broken pavement allowing them to rock in seesaw fashion as traffic passed over them. In other instances, where the pavement had become broken into small pieces of about 6 to 12 in. across, they were loose and moved with the passing of the traffic.

One such rigid pavement over an 8-mi project became so rough in 1949 that it required rehabilitation, and a study was made as to what method should be chosen for rehabilitating it. Because the project showed a considerable number of rocking slabs and numerous places where the pavement was broken into small pieces which were loose and moving under the traffic, it was decided to require the use of a pavement breaker in preparing the places where the old pavement showed rocking slabs and broken sections.

This was set up in the contract as breaking and reseating concrete into the subgrade. The special provisions to accomplish this required that the breaking of the concrete should be done with a drop hammer (steam, gasoline, or electric power), pneumatic hammer and other equipment suitable to perform the work required. After the pavement had been broken to the specified dimensions of an area not to exceed 1 sq ft with the maximum length of any one side not greater than 18 in., the pieces were required to be compacted and seated by rolling or by the use of mechanical tamps or other approved methods. The roller, if used, was to be rated at not less than 10 tons. The compaction and seating was to be continued until the entire area of broken pavement had been firmly bedded and there was no preceptible disturbances of the broken pieces

Paper sponsored by Committee on Salvaging Old Pavements by Resurfacing.

Figure 1. Old, broken, and rough rigid pavement.

TABLE 1

GRADATION OF FINE AGGREGATE

Sieve Size	Percent Passing
No. 3	100
No. 4	80 - 100
No. 50	10 - 40
No. 100	0 - 10

ahead of the roller or under the mechanical tamps and there should be no rocking of the pieces or evidence of further settlement.

During the rolling or tamping of the broken pavement, sand or screenings of the gradation (Table 1) were to be applied gradually to the surface and swept into the crevices ahead of the compaction equipment.

The compaction and application of the sand or screenings was to be continued until all cracks and voids had been filled and the entire course was thoroughly compacted and uniform. The low bid on this item of breaking and reseating concrete into the subgrade was $0.25 per sq yd.

During the breaking and reseating of the pavement, the results did not turn out as had been planned. The subgrade was so well compacted and firm that in trying to seat the small pieces into the subgrade, they became further broken causing such a swell in the thickness of the course that part of the broken pavement had to be removed to hold to the pre-established finished grade.

With this experience on two or three sections of the project, the operation was changed whereby the rocking pavement slabs were broken just enough to seat the slabs on the subgrade reasonably well. This method appeared to be a more satisfactory way than the pounding of the pieces of pavement and trying to make a base course of the pieces and sand or screenings.

After the old pavement had been prepared by this latter method and made stable, and after the base widening had been placed, a leveling course of hot asphaltic concrete mixture was placed and then followed by $2\frac{1}{2}$ in. of penetration macadam, and $1\frac{1}{2}$ in. of hot asphaltic concrete binder and 1-in. surface mixture.

Within a few months this project showed reflection cracks in the resurfacing along the joints and larger cracks. After 12 years of service, the reflection cracks had further developed along the cracks formed by the small broken pieces.

Since this experience, it is very seldom that the large pavement breaker is used. However, occasionally when a rocking slab is found and it is not broken into small pieces, a 2-ton pile-driving hammer operated from a small portable crane is used before resurfacing. The hammer is dropped only 4 to 5 ft and with a minimum of blows to seat the slab on the subgrade. Although this is not considered as producing a perfect job, it is an economical way of using old rocking slabs for an extended period.

The general practice for the last few years is to remove the broken pieces of a foot or two in area and replace for the full depth with either asphaltic concrete or a soil-stone mixture stabilized with portland cement. This latter mixture is a very satisfactory and economical material for both a full-width base and for widening old rigid pavements. The mixture produces a semi-rigid slab which after hardening seldom becomes distorted along the joint between the old and new. Many of the mixtures never produce shrinkage cracks; however, others do and they are usually reflected through the overlay surfacing.

The design of the mixture and the construction procedure of placing the mixture is a relatively simple one. When a project is set up for rehabilitation, a survey is made as to the availability of local soils suitable for cement stabilization, and when found, an option is secured on the deposit and information concerning the size, quantity of material, clearing and grubbing, royalty, price, and other features are furnished to the contractors for bidding purposes. The design is simply a requirement that the final mixture contain a minimum of 60 percent by weight of aggregate retained on the No. 10 sieve and passing the 1½-in. sieve. The remaining 40 percent or less must be a suitable, friable type of binder material that can be easily pulverized and mixed with portland cement and that will become hardened or set after the cement is added. The binder may be produced with screenings, sand, and clay or sand clay or topsoil, whichever may be available. If economical, a straight soil-cement mixture is sometimes used.

The method of arriving at the percent of cement is to predetermine a strength of 300 psi at 28 days or 225 at 7 days by triaxial testing with 20-lb lateral pressure. This strength requirement results in cement content of around 2 to 3 percent by volume with the stone mixture and 6 to 11 percent with the soil-cement.

The construction procedure is that the broken pavement is taken out for the full depth, the subgrade prepared by shaping and rolling or pneumatic tamping on the surface and if the project is widened, the shoulders are excavated to a depth of about 8 in. and a special flat steel-wheel roller (Fig. 2) is employed to compact the shoulder or trench subgrade on each side of the pavement.

The cement-stabilized stone mixture is prepared in a portable mixing plant set up at a suitable location on the project; all ingredients consisting of coarse aggregate, binder, cement, and water are mixed, transported to the project, and spread by specially constructed spreaders (Fig. 3) for the widening and the ordinary type of spreader for the full section after which the mixture is compacted by rolling (Fig. 4) to 100 percent maximum density (AASHO Method T 99). After compaction, the surface is finely graded and rerolled to the finished grade (Fig. 5). These operations are followed

Figure 2. Compacting trench subgrade.

Figure 3. Specially constructed spreader for widening.

Figure 4. Compacting mixture.

Figure 5. Rerolling surface to finished grade.

Figure 6. Bituminous prime coat.

Figure 7. Placing hot asphaltic binder.

very closely back of the spreading. The average prices for the soil-bound macadam, cement-stabilized mixture, in place, is $2.25 per ton with the cement having a price of $4.90 a barrel.

After the material has been placed, on the following day a bituminous prime coat is applied (Fig. 6) at the rate of 0.15 to 0.20 gal per sq yd. The bituminous prime coat acts as both a curing agent and a prime coat. After about 7 days of curing, the surface wearing course is then placed over the entire pavement.

This wearing course usually consists of a leveling course of hot asphaltic concrete, followed by hot asphaltic binder (Fig. 7) and surface course mixture. The penetration macadam used in the older project has been found to be somewhat unstable under present-day heavy traffic in that movement apparently takes place within the course causing reflection cracks in the surface courses which soon begin to show deteriorating crack patterns. However, on the old project already mentioned, the displacement of the penetration macadam, under the heavy traffic could not be differentiated between that and the movement of the old broken-up rigid pavement. But, experience on other projects has shown the penetration macadam is not as stable as asphaltic concrete; therefore, the later procedure has used asphaltic concrete overlays on rigid pavements with the thought in mind that there might possibly be some movement within the penetration itself.

In rehabilitating rigid pavements it is concluded from the work done in Georgia; (a) that a pavement breaker is only suitable for use in very few instances; (b) that if the pavement is broken too badly, then it should be removed for the full depth and replaced; and (c) that the cement-stabilized stone mixture has proven to be an economical material for use in replacing and widening rigid pavement.

A New Approach to Subsealing

T. V. FAHNESTOCK, Bituminous Engineer, North Carolina State Highway Commission, Raleigh; and
R. L. DAVIS, Technical Representative, Koppers Company, Inc., Pittsburgh

This paper describes the development of an asphalt emulsion for subsealing concrete pavement. The asphalt emulsion contains a micro-aggregate or filler that makes the residue from the broken emulsion a mastic. This mastic is too stiff to be squeezed through the joints or cracks in the concrete. The asphalt emulsion subsealer is applied cold with standard bituminous equipment and is successfully used in damp or wet areas. This material does not "track" on the pavement if any spillage is allowed to dry before traffic hits it. Several years of experience in subsealing with this asphalt emulsion in North Carolina indicate that it is a safe, economical, effective subsealer for concrete pavement.

•SEVERAL division engineers with the North Carolina Highway Commission requested about twelve years ago a cold-applied asphalt emulsion suitable for subsealing cement concrete pavement. These engineers wanted especially a cold-applied material because several men in the subsealing crews had recently been burned while subsealing with a hot-applied material. There was no asphalt emulsion available at that time considered suitable for this application. However, the division engineers considered the hot application so hazardous and so hard on equipment that some of them began using a regular, high viscosity quick breaking or RS-2 type emulsion for subsealing. The results of this operation turned out surprisingly well in several instances, but there were also cases where some of the asphalt residue from the emulsion was squeezed back up through the joints or cracks in the cement concrete. This material was "tracked" down the highway because the emulsion base was soft and sticky at summer temperatures.

A hot-applied material was still the only subsealing material approved by the office of the bituminous engineer. However, after the loss of a new bituminous distributor and another bad accident which nearly caused the death of two men, a program was undertaken to develop a subsealing material that could be applied cold.

DEVELOPMENT OF COLD-APPLIED SUBSEALER

The first efforts in the development of a cold-applied subsealer were in the direction of changing the method of applying the RS-2 type of emulsion with the idea of keeping it from coming back through the pavement. Although these efforts were at least partially successful, it was decided that a subsealer with an appreciable yield strength was desirable.

An attempt was next made to pump a mixture of slow-breaking emulsion and stone screenings under the pavement using a mudjack. This operation was abandoned because sufficient back pressure was developed to stall the mudjack. In fact, the mixture was packed so tightly that it was necessary to disassemble the mudjack completely in order to clean it out.

Next, an asphalt emulsion was made in which a large percentage of portland cement was incorporated. This material was designed to leave stiff mastic residue under the concrete pavement. Though the emulsion was quite fluid, some difficulty was experienced in pumping it with the usual bituminous pumps. However, it could be pumped easily by a centrifugal pump. Even though this asphalt emulsion containing portland

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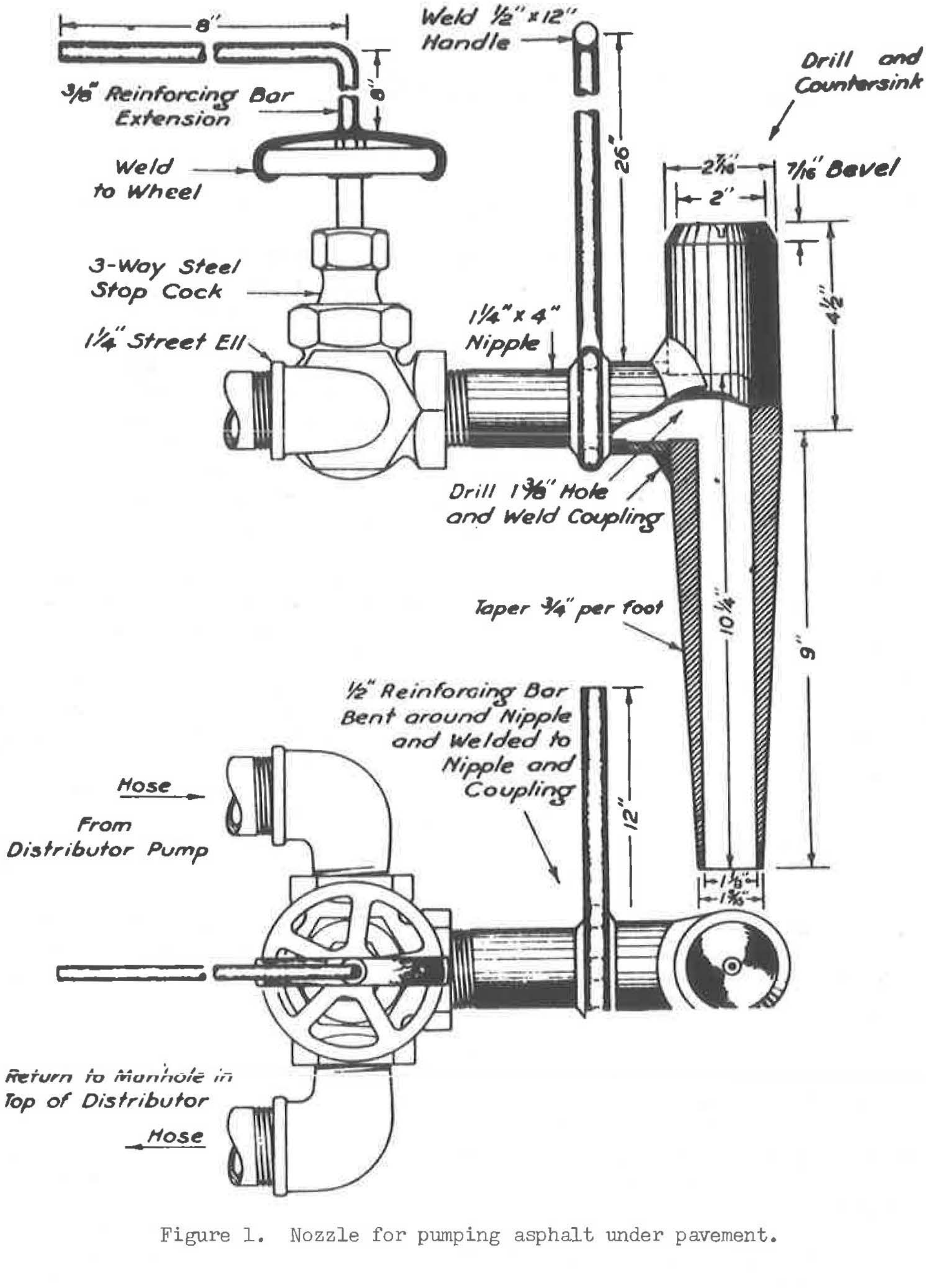

Figure 1. Nozzle for pumping asphalt under pavement.

cement seemed to subseal satisfactorily, there were some problems in its use. The manufacturer sold it only in drums, because it was feared that it might solidify if stored for extended periods. These fears proved to be well founded because some drums of this material which were left over at the end of the season were completely set up by the next spring. When the drum was stripped from around the emulsion, the contents looked like a dark cement concrete drum.

To avoid the possibility of having this asphalt emulsion set up in equipment and to reduce the high cost involved in drum shipments, another asphalt emulsion was made containing a less active micro-aggregate or filler. This material was called Subseal-

Figure 2. Pumping of subsealer.

Figure 3. Wooden plug being driven into slab.

er II, and it was judged safe to ship in tank trucks or tank cars. Subsealer II could be pumped using standard bituminous pumps.

METHOD FOR APPLYING SUBSEALER II

Subsealer II is easily pumped under the concrete and dries to a stiff residue or mastic. The established practice in North Carolina is to drill $1\frac{5}{8}$-in. holes at the selected points for subsealing. In the shoulder at the joint or crack being subsealed, a hole is dug which extends an inch or more below the underside of the pavement slab. Compressed air is forced through the $1\frac{5}{8}$-in. holes in the pavement to blow as much mud and water from under the pavement as possible. The holes dug in the shoulder make it easier for the water and mud to be blown from under the pavement, and also permit observation of when the asphalt emulsion reaches the edge of the pavement. These holes in the shoulder are to be filled with dirt and tamped after the asphalt emulsion reaches them.

After blowing out the $1\frac{5}{8}$-in. holes in the pavement, a nozzle tapered to fit tightly is driven into the pavement. Sand is spread in a 12-in. radius around the nozzle. The sand prevents any asphalt emulsion that may leak around the nozzle from sticking to the pavement. The Subsealer II at ambient temperature is then pumped from a 600-gal utility kettle through the tapered nozzle and under the concrete slab. Figure 1 shows the nozzle, and Figure 2 the subsealer being pumped. A 10-ft straightedge is used to determine when the slab is at the right elevation. The man watching the straightedge should signal the valve operator when the slab begins to rise. The elevation of the slab is then carefully adjusted to the proper level. The nozzle should be left in the pavement for about 30 sec before being withdrawn. This is to allow time to see if the slab is going to maintain the proper level. If the proper level is maintained, the nozzle is withdrawn from the hole and a tapered wooden plug is immediately driven into hole. Figure 3 shows the wooden plug being driven into the slab. The sand around the hole and any asphalt emulsion that has gotten on it while removing the nozzle are scooped up with a shovel and removed from the pavement.

A subseal of any kind seldom cures the movement of concrete slabs forever. This is true of subsealing with Subsealer II. Though in some instances one subsealing will cure the movement for many years, the more usual case is that it is necessary to subseal again in two to three years even though some permanent improvement will be maintained.

One of the most desirable features of Subsealer II is the elimination of many of the hazards which were part of the subsealing operation in the past. This material is usually applied with no heating at all. It is not inflammable or explosive, and it is very easy to handle. The experience in North Carolina is that it is not as hard on equipment as materials used in the past.

EFFECTIVENESS OF SUBSEALER II

The first work that was done with Subsealer II was in Craven County, in 1959. This work was done in areas which were originally swampy and where water was a problem. Even after blowing out the drilled holes in the pavement with compressed air, there was a considerable amount of free water under the slab. It was found that quite a bit more water could be forced out when the Subsealer II was pumped under the slab. The practice was to continue to pump until the material flowing out at the edge of the pavement appeared to be undiluted Subsealer II (Fig. 4).

Examination of this work a year later showed that it had been effective in reducing slab movement or pumping at all locations where it was used. At some points, there was still some evidence of pumping but at a much reduced level; at many others, the pumping appeared to have been arrested. At no point treated was there any indication of extrusions of the Subsealer II. The over-all judgment was that Subsealer II was as effective as any other subsealer that had been used in the past (Fig. 5).

Subsealer II has been used each year since 1959. Its use is increasing in North Carolina, and more than 150 miles of roadway have been subsealed in various parts of the State. After nearly four years, the subsealing done in 1959 continues to be satisfactory.

Figure 4. Undiluted Subsealer II being exuded from edge of pavement.

Figure 5. Road with subsealer.

There have been some requests for cost per mile figures on this type of subsealing. Estimates of this type have not been included, however, because the cost per mile varies almost directly with the number and volume of cavities to be filled and bears little relationship to the distance covered. However, the original cost of Subsealer II is about the same as for other subsealing materials, but it is more economical to apply. This makes the over-all cost less than materials used in the past in North Carolina.

Several years experience in using Subsealer II confirms that it is a safe, economical, and effective subsealer for concrete pavement.

ACKNOWLEDGMENT

Figure 1 is reproduced by permission from Vol. 16 of the Proceedings of the Association of Asphalt Paving Technologists (1947).

Appendix

Equipment

1. A 600-gal utility kettle or pressure distributor (Fig. 2).
2. A 3-way valve.
3. A $1\frac{5}{16}$-in. tapered to 2-in. (outside diameter) nozzle to insert in the pavement and necessary hose for line from nozzle to three-way valve and double line from three-way valve to tank (Fig. 1).
4. A 105-cfm air compressor and hand held drill with $1\frac{5}{8}$-in. carbide bit.

Subsealer II Specifications

1. Residue by evaporation, 45 to 60 percent.
2. Water, 40 to 55 percent.
3. Ash on the basis of non-volatile material, 40-60 percent.

Effect of Pavement Breaker Rolling on Crack Reflectance in Bituminous Overlays

PAUL G. VELZ, Hydraulics Engineer, Materials and Research Section, Minnesota Department of Highways

• THE USE of a heavy roller to crack old portland cement concrete pavement and to seat it on the subgrade before resurfacing with bituminous mixtures was tried experimentally in Minnesota in 1959. The project is located on T.H. 212 between Bird Island and Stewart in central Minnesota. It is a typical widening and resurfacing project, in which the bituminous mixtures were placed directly on the old pavement.

A report on the project was made in 1961 which included detailed information on such items as the design of the old pavement, the size of the roller, the immediate effects of rolling on the pavement, typical sections, costs of the reconstruction, and performance of the project during the first six months. This report was published in Highway Research Board Bulletin 290.

Briefly, some of the pertinent items can be summarized as follows. The concrete pavement was 28 years old, having been constructed in 1931. It was 20 ft wide with a 9- by 7- by 9-in. cross-section. Panels were generally 40 ft 4 in. long with every other joint an expansion joint. All the joints were doweled, and the panels were reinforced at the edges, along centerline and along the joints.

The entire project was 18 mi long, of which only 1½ mi were rolled. In this 1½-mi experimental section, each lane was covered by ten passes of a 59-ton roller which produced a great number of transverse cracks in the old pavement. The roller had four wheels on a single axle, and the tire air pressure was 90 psi. Actual pressures on the slab averaged 83.5 psi as determined from the gross contact areas of the tires.

The rolling caused 1,868 new cracks in the two lanes of the pavement, and nearly all of these were transverse cracks. When the old cracks and the joints were included, there were nearly 3,000 openings in the pavement. In other words, there was a transverse crack or a joint about every 5 ft after rolling.

The bituminous mixtures for widening and resurfacing were all hot-mixed. The typical section on the project consisted of a 1½-in. wearing course, a 1½-in. binder course and a 1½-in. leveling course for a total of 4½ in. of bituminous mixture. However, on the rolled section, three different thicknesses were used (namely, 5, 6, and 9 in.); the difference being in the thickness of the leveling course. Therefore, the pavement that was rolled was divided into three test sections corresponding to the variations in the thickness of the overlay. For comparison purposes, three control sections were selected in the unrolled portion of the project where the overlay was 4½ in. thick.

PROJECT PERFORMANCE

Since completion of the project in October 1959, seven crack surveys and several rutting and roughness measurements have been made to evaluate the performance of the test sections over the 2½-yr period following construction. The results of these studies and the conclusions drawn from them are reported in this paper.

The crack survey data have been reduced to curved line graphs for the purpose of showing the general trends of crack progression with age. This required the striking of averages to establish certain portions of the curves, particularly at the early ages. However, the curves have been drawn to fit as many points as possible, and generally the terminal points of the curves at the age of 30 months coincide with the survey data.

Paper sponsored by Committee on Salvaging Old Pavements by Resurfacing.

Transverse Joint Reflectance

Nearly all of the transverse cracking in the portion of the bituminous surface over the old pavement has occurred at the joints. Figure 1 shows the progression of joint reflection with age. The dashed lines for sections 1, 2, and 3 represent the unrolled control sections with 4½ in. of bituminous overlay, and the solid lines represent the rolled sections with 5, 6, and 9 in. of bituminous overlay.

In section 1, 98 percent of the transverse joints reflected as cracks in the bituminous surface almost immediately after construction. The other two control sections reacted very much alike, progressing to about 90 percent joint reflection in 18 months and to 95 percent in 30 months.

In contrast to this rapid reflection of joints, the 5- and 6-in. rolled sections had about 50 percent reflection in 18 months and 75 percent in 30 months, a sizable decrease in the number of cracks that would require maintenance. This reduction in joint reflection is largely due to the effects of the rolling. The 9-in. rolled section showed the least cracking over the joints, reaching only 60 percent in 30 months. The difference between this 9-in. section and the other rolled sections is probably due to the additional thickness of the bituminous overlay.

Centerline Joint Reflectance

Figure 2 shows the progression of cracking over the centerline joint in the old pavement. Here again, section 1 stands out as a poor performer, with centerline cracking starting immediately after construction and progressing to 94 percent in 30 months. In comparison, centerline cracking did not become significant during the first 12 months on all the other test sections. However, sections 2 and 3, which were not rolled, were about 90 percent cracked on centerline at an age of 30 months.

The 6- and 9-in. rolled sections had very similar amounts of centerline cracking, reaching 70 to 75 percent at 30 months. The 5-in. rolled section was the slowest to start cracking; but, at 30 months, had nearly the same amount of centerline cracking as the unrolled sections, reaching 87 percent.

These data indicate that rolling had a slight beneficial effect in retarding cracking over the centerline joint in the old pavement. The effect of the thickness of the bituminous overlay does not seem to be well defined, though at 30 months the 9-in. rolled section had the least centerline cracking.

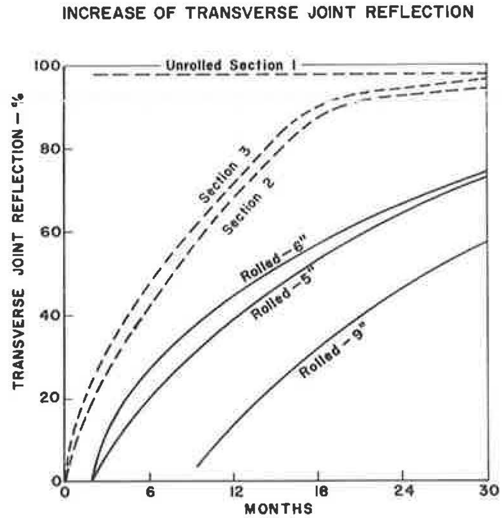

Figure 1. Progression of reflectance cracks over joints in the old concrete pavement.

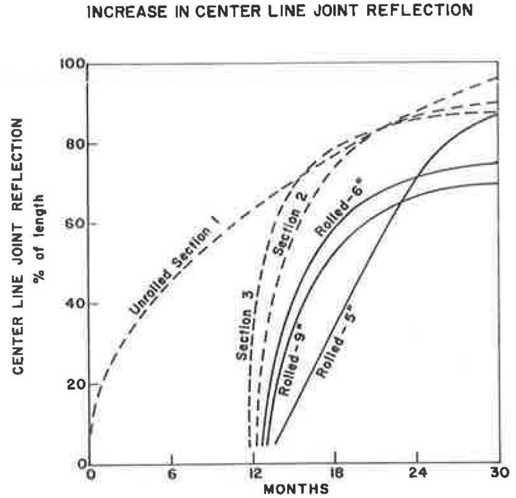

Figure 2. Progression of centerline joint reflection cracking.

Transverse Cracks in the Widening

Contrary to what one might think, pavement breaker rolling had quite an effect on the cracking in the bituminous widening outside the edges of the old pavement. Figure 3 shows the progression of transverse cracks across the widening strip by plotting the average feet of cracks per station against the age of the project in months.

Section 1 again showed a tendency to crack sooner than the other sections, but section 3 surpassed section 1 at 30 months with 38 ft of cracks per station as compared to 34 ft per station.

Section 2 started cracking soon after construction, like the other unrolled sections, but only progressed to 16 ft per station at 30 months, about the same as the rolled sections. However, the rolled sections did not start to crack until after 6 months for the 6-in. section and after 12 months or more for the 5- and 9-in. sections. The 9-in. section was the slowest to develop transverse cracks in the widening; but, at an age of $2\frac{1}{2}$ yr, this thick section was no better than thinner sections with $4\frac{1}{2}$ and 5 in. of bituminous overlay.

One might conclude, therefore, that rolling was beneficial in retarding transverse cracking in the portion of the bituminous surface outside the old concrete pavement, and that the thickness of the overlay was not particularly significant in this case.

A number of the transverse widening cracks occurred without relation to cracks or joints in the old pavement. This so-called random cracking was quite variable among the sections, ranging from 4 to 26 percent of the total. Generally, there was more random cracking in the sections with the most cracks.

Longitudinal Widening Cracks

There is one more type of crack that is typical of widening and resurfacing projects such as this, and that is the longitudinal crack which generally develops at the edge of the old pavement. Figure 4 shows the progression of this cracking by plotting the percent of the total length cracked against age in months.

Section 1 was the first to develop longitudinal widening cracks, being 20 percent cracked in 12 months and 29 percent in 30 months.

Section 3 started to crack along the edge of the old pavement at 6 months and progressed quite rapidly to 40 percent at 30 months. Indications are that this section will continue to crack at a fairly rapid rate.

Section 2 was the last of the unrolled sections to develop widening cracks, starting about one year after construction and being 17 percent cracked at 30 months.

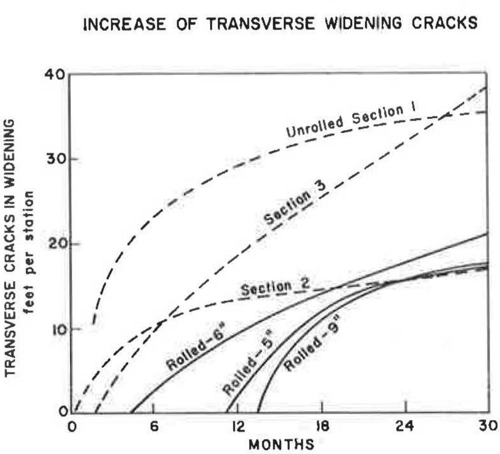

Figure 3. Progression of transverse cracks in widening.

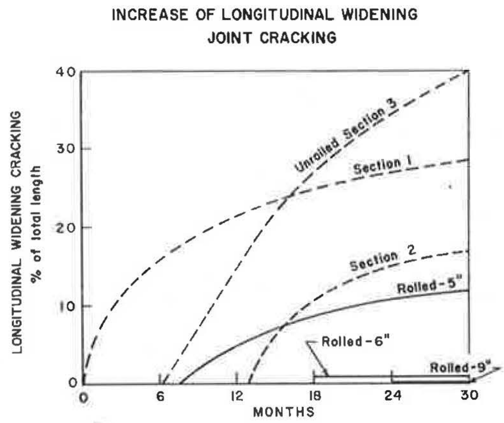

Figure 4. Progression of longitudinal widening cracks at edge of old pavement.

The 5-in. rolled section started cracking at about 7 months and progressed to 12 percent at 30 months. The other two rolled sections have practically no longitudinal widening cracks—only 1 percent for the 6-in. section and none for the 9-in. section in $2\frac{1}{2}$ years. This is quite a remarkable performance considering the clay soils in the subgrade, the prolonged freezing, and subsequent severe spring breakup periods experienced in Minnesota. Apparently, the combination of the rolling with the thicker overlays was effective in controlling longitudinal widening cracks.

Rutting in Wheel Tracks

Cross-section measurements on the bituminous surface have been made on six occasions over a 2-yr period following construction to determine distortion or rutting in the wheel tracks. These measurements were made at three locations in each test section, except section 3.

In the unrolled sections with $4\frac{1}{2}$ in. of bituminous overlay, rutting of less than 0.25 in. was found in section 2 and none in section 1.

In the rolled sections, rutting up to 0.25 in. was found in the 5- and 6-in. sections and none in the 9-in. section. This would indicate that the thickness of the overlay was not a contributing factor.

Because traffic on this highway is not particularly heavy, no serious rutting was expected in this short a period.

Roughness

Roughness measurements were made on five occasions since completion of the project in 1959, the latest readings being made in April 1962. The Minnesota roughometer is similar to the AASHO road roughness recorder, and produces roughness index values expressed in inches per mile.

After construction in 1959, the project averaged 56 in. per mi which increased to 60 in. per mi in April 1960 and to 64 in. per mi in April 1962. Using the April values for comparison purposes, thereby eliminating seasonal effects, the project average increased 4 in. per mi during the 2-yr period ending in April 1962. During this same period, sections 1 and 2 increased 6 in. and 9 in. per mi, respectively, and the rolled sections increased 6 in. per mi.

The roughness indexes on all the sections fell within the range of 62 to 68 in. per mi in 1962, which are very acceptable values for a bituminous surface. Apparently the project is not old enough to show significant changes in roughness.

CONCLUSIONS

The general conclusions which may be drawn from the information obtained on the T.H. 212 project are as follows:

1. Ten passes of a 59-ton roller will produce an optimum amount of cracking in old concrete pavements at a reasonable cost.
2. Pavement breaker rolling will retard the development of cracks in the bituminous overlay above the pavement and in the widening for periods longer than $2\frac{1}{2}$ yr.
3. Thicker bituminous overlays will increase the retardation effect of the rolling, but the increased cost of very thick overlays may offset the additional benefits.
4. No significant rutting in the wheel tracks will develop in overlays up to 9 in. thick under $2\frac{1}{2}$ yr of average rural traffic in Minnesota.
5. No significant change in roughness should occur for $2\frac{1}{2}$ yr after construction.
6. Because the experimental rolled sections on the T.H. 212 project showed a definite reduction in cracking, the Minnesota Highway Department has used pavement breaker rolling on an 8-mi resurfacing project in 1962.

Slab Breaking and Seating on Wet Subgrades With Pneumatic Roller

JAMES W. LYON, JR., Testing Engineer, Louisiana Department of Highways

This paper reports on a long-range field study to determine the practicability of using a pneumatic tire 50-ton roller to break and seat old concrete pavements before overlaying with hot-mix asphaltic concrete. The method of study is a comparative analysis of the behavior, under traffic, of various roadway sections employing different breaking and seating techniques of the 50-ton pneumatic roller and an impact hammer on a construction project generally having a wet subgrade condition.

Observed results and field data are shown during the breaking and seating operation and after one and two years of traffic to support the conclusion that the 50-ton roller should be used in conjunction with an impact hammer, using three roller coverages for slab breaking and seating.

• FOR SEVERAL years the Louisiana Department of Highways Testing and Research Section has been interested in better construction methods using bituminous hot mix to overlay and renew old surfaces of which old concrete pavements are the major portion. After overlaying, a frequent problem is that slab movement and pumping have continued. Realizing the need for better concrete slab breaking and seating construction techniques, the Testing and Research Section has undertaken the task of determining the feasibility of using a high-intensity roller to prepare old concrete slabs for hot-mix overlaying. This study was undertaken on a construction project, using various treatments of a 50-ton roller in conjunction with the present impact hammer method of slab breaking and seating.

The project site being used for this study is 15¾ mi of old, badly pumping, 18-ft wide concrete pavement, comprising State Route 1 between Donaldsonville and Napoleonville, in the bayou country of south Louisiana (Fig. 1). The existing concrete slab is 6 in. thick at the center, thickened to 8 in. at the edges, and widened 3 ft on each side with concrete. It was overlaid in the winter months of 1959 with hot mix, after various slab breaking and seating techniques with the 50-ton roller and the presently used impact hammer. The hot-mix overlay was 3½ in. of binder and wearing course, with a quantity of binder course for leveling, equivalent to 1 in. of thickness (Fig. 2).

The average daily traffic on this project is approximately 3,000 vehicles per day; approximately 18 percent of this is trucks with 2 percent having axle loads in excess of 18,000 lb.

The subgrade soils immediately under the old slab are silty loams, silty clay loams, and silty clays containing more than 50 percent silt. These soils generally have moderate plasticity indexes, varying from 5 to 15, with optimum moistures ranging from 14 to 20 percent, and with group indexes of from 8 to 12. These soils would generally classify as ML and CL by the Unified Soil Classification System.

The concrete pavement was generally unreinforced (Fig. 3). Transverse, ½-in. round by 4-ft long, deformed bars, spaced 5 ft on centers, were employed along the longitudinal center joint. Transverse expansion joints consisted of a premolded joint filler and eight ¾-in. smooth, round dowel bars, 4-ft in length for load transfer. The

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general spacing of expansion joints was approximately 500 ft.

The impact hammer used for seating and breaking in this study was truck mounted, with a 4,000-lb dropping head. The 50-ton roller was a Bros model 450, with four 16.00-21 36-ply tires, mounted in line, and was towed by a Euclid prime mover. The roller was loaded to a gross load of 48 tons with sand and water. The tire pressure was 90 psi. The gross roller load and tire pressure were kept constant at all times in the study.

Economically, the use of the roller compared very favorably to the use of the hammer, per mile of roadway treated. The contract amount for concrete seating and breaking with the hammer was \$15 per hr of actual operation. The amount for rolling with the 50-ton roller was \$25 per hr. The impact hammer cost approximately \$88.60 per mi treated. The roller cost approximately \$90.80 per mi rolled.

For the purpose of this study, three roadway sections were chosen, with each section containing portions representative of the total project. The length of these sections varied from 3½ to 5 mi each. One test section was established as a control, with slab seating and breaking using the impact hammer. Another test section used the 50-ton roller for slab breaking and seating. The third test section used the roller and the impact hammer for slab breaking and seating. Within this third section, the general procedure was to locate the moving slabs with the roller, break with the drop hammer, and then seat the broken slabs with the roller.

From four to eight test stations were established within each of the three sections. These test stations were used for collecting field data, representative of their respective section.

Figure 1. Typical slab pumping under truck traffic on old roadway before construction.

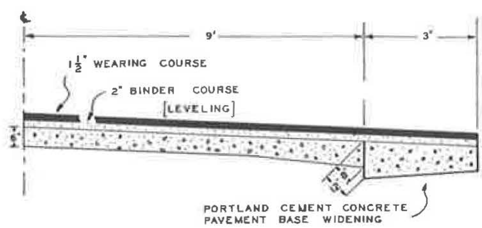

Figure 2. Typical section, as constructed.

Figure 3. Old roadway showing pumping under normal traffic; 3-ft base widening in place, before any breaking and seating operation.

ACCUMULATION OF FIELD DATA

The basic consideration of this study is to detect continuing slab movement, its location, and its extent. To accomplish these aims, comparative data are being obtained on the various research treatments, taking surface roughness and deflection, and observing reflection cracking. With this in mind, the Benkelman beam, the road roughness recorder, and visual surveys, along with elevation checks for over-all subsidence are being used as prime study tools.

The Benkelman beam is a device that detects and measures movement of the roadway under a given load. In this study the axle load used was 18,000 lb (the Louisiana single-axle legal load limit), and was applied with a Department maintenance dump truck loaded with sand. The pavement deflections were measured to 0.002 in. under this load. The Benkelman beam deflections reported in this study are relative, and not reported as an absolute measure of movement, as the beam itself was probably influenced by the movement of the 18,000-lb wheel load.

The L.D.H. road roughness recorder (an adaptation of the BPR single-wheel unit) measures an accumulation of vertical movement per the unit length of roadway under study. A change in this vertical movement, or roughness, would indicate moving slabs. Whereas the Benkelman beam directly measures the movement of the pavement at a point on the roadway, the roughness recorder is an accumulation of movements over a length of roadway. In effect, the two devices tend to complement each other, giving a relatively complete picture of movement over the entire roadway length under study.

In addition to the deflection readings and roughness data, moisture content data at 1- and 2-ft depths beneath the concrete pavement have been gathered, and water table variations have been measured in cased wells extending to an elevation at least 6 ft below the finished overlay grade. This is to determine the effects of moisture content variation on slab movement in conjunction with the various breaking and seating operations.

A culvert survey has been completed on the in-place cross-drains to determine the effects of the 50-ton roller.

FIELD ROLLER RESULTS

In the first phases of the roller operation, the roller speed was varied from creep speed to 5 mph. It was found that a roller speed of 2 mph gave the best results. This speed was then maintained throughout the rest of the study for locating moving slabs, breaking, and seating. The roller can be an awkward piece of equipment, in that it is not very maneuverable on a restricted roadway. It cannot be turned around at will, and road intersections generally must be used for turning. In this study, this possibly caused overrolling just to move the roller where it was needed. It appears that the maximum effective length of roadway that could be rolled without excessive personnel fatigue was 2 continuous miles.

In spite of the heavy weight, there was no displacement or noticeable damage to the cross-drains, even though approximately one-half of the culverts had an overlying cover of from 10 to 18 in., including the old slab.

Rocking slabs can definitely be created or their rocking intensified, if the subgrade is wet and the slabs are overrolled with the 50-ton roller. Very little slab breaking was noted where the roller was used on a wet subgrade. From preliminary data it appears that the roller must be used with extreme caution wherever the in-place moisture contents exceed the optimum moisture by about 5 percent or more. Wherever the subgrade was rolled with a moisture content nearer optimum, the roller broke most of the rocking slabs transversely at or within the third points. Occasionally, slabs were broken longitudinally, again, at or within the third points. The slabs broke with a distinct popping and grinding sound; however, the breaks generally were not visible until the following pass. Where the old pavement had been badly broken under traffic, the roller did little to break the slabs further.

Apparently six or more coverages of the 50-ton roller tend to create rocking slabs on this particular type of subgrade, regardless of moisture content. Generally, slab breaking was accomplished in the first two coverages of the roller, with the majority of the breaking occurring on the first coverage. Moving slabs are easily detected with one pass of the roller. The use of a diamond saw to score the rocking slabs before rolling did not appear to facilitate the breaking of the slabs. In addition, the sawing of the slabs was slow, and impractical for relatively long sections of roadway.

The 3-ft base widening definitely interferes with the lateral movement of any underlying wet soil slurry. The roller forces this material through the joints or breaks in the slab and along the edge of the slab when the existing shoulder or the 3-ft base widening was in place. The majority of this project was studied with the base widening in place (Figs. 4, 5, and 6).

Figure 4. Wet material displaced by rolling at typical pumping slabs.

Figure 5. Wet material removed by roller from subgrade under slabs broken by traffic; removed mud in trench.

Figure 6. Wet subgrade material forced through shoulder by roller.

The 50-ton roller is a relatively fast and thorough piece of equipment for breaking and seating. The roller generally covers the entire lane width being treated. This, in effect, gives more complete slab breaking and seating than that of the impact hammer, which is generally a spot means of breaking and seating. It appears that the presently used impact hammer method of slab breaking breaks and partially seats the old slabs, but traps a major portion of the underlying soil slurry. The roller appears to remove a greater proportion of this soft material, inferring better seating.

Subgrade soil conditions should be considered in determining the gross roller weight and the tire pressures to be used. The roller should be weighted to an amount that would give the desired results without damaging the subgrade. The least number of coverages that will give the desired results should be used. The possibility of overrolling and the location of the turnarounds should be considered in developing rolling sections. Additional passes just to move the roller to various locations should be avoided.

The use of the roller by itself does not inhibit reflection cracking. However, the use of the hammer together with the roller at its optimum number of coverages does drastically reduce reflection cracking. No pumping joints have been noticed in any of the three test sections.

For improved results, using the roller for breaking and seating, field observations indicate that a narrow trench should be opened adjacent to the slab, extending below the bottom of the slab. After rolling with the 50-ton roller, a leveling course of hot mix should then be laid on the old slab. The trench cut to facilitate the removal of soft material should then be closed. The base widening could then be placed to the grade of the leveling course previously applied. The binder and wearing courses could then be laid over the leveling course and base widening.

GENERAL RESULTS

The entire length of this project has been closely studied for signs of slab pumping. After two years of service, there are no physical evidences of any pumping in any test section, whether rolled, hammered, or treated with a combination of rolling and hammering.

Maximum and minimum deflections, as measured by Benkelman beams, before slab seating and at one year and two years after overlaying, are given in Table 1. The deflection study, on this research project indicates that a relative slab movement of 0.040 in. or less is necessary for a continued pavement life without excessive maintenance and without a loss of riding qualities.

Sections 1, 2, and 3 had good areas before overlaying, as indicated by the minimum deflection values, and bad areas having more than 0.040 in. of movement, as indicated by the maximum deflection values. At one year after overlaying, the deflections are very consistent, being within a range of 0.005 to 0.014 in., and the highest deflection within any section, is equal to, or less than, the lowest deflection before breaking and seating operations. The greatest deflection changes from before seating to one year after overlaying are in sections 2 and 3, particularly the reduction of the maximum deflection values of section 3.

The average maximum deflection of section 3 for two years after overlaying has not increased from that of one year after overlaying. Whereas, there is some increase in these deflections for sections 1 and 2.

Figure 7 shows the effects of roller coverages on the subgrade moisture contents at 1- and 2-ft depths below the concrete pavement expressed as a percentage of the moisture content immediately after rolling, as compared to that just prior to rolling. Two roller coverages appear to draw moisture from the 2-ft depth into the 1-ft depth in an amount that would offset the moisture removed from the 1-ft depth. Three roller coverages appear to be more effective in subgrade drying, removing relatively more water from either the 1- or 2-ft depths than that pumped up into these layers. Four roller coverages apparently pump water into the 1-ft depth faster than it is removed. Also, four coverages probably begin to draw water from the deeper subgrade into the upper 2 ft of the subgrade. Six roller coverages were observed during breaking and seating operations, and field observations indicated that slab pumping was intensified under this number of coverages. Although the 50-ton roller undoubtedly created a

TABLE 1

AVERAGE MAXIMUM AND MINIMUM DEFLECTIONS

Section	Method	Avg. Deflections (in.)					
		Before Seating		1 Yr After Overlay		2 Yr After Overlay	
		Max.	Min.	Max.	Min.	Max.	Min.
1	Hammer only	0.085	0.016	0.010	0.005	0.013	0.008
2	Roller only	0.122	0.018	0.014	0.008	0.019	0.008
3	Hammer and roller	0.154	0.012	0.013	0.006	0.013	0.009

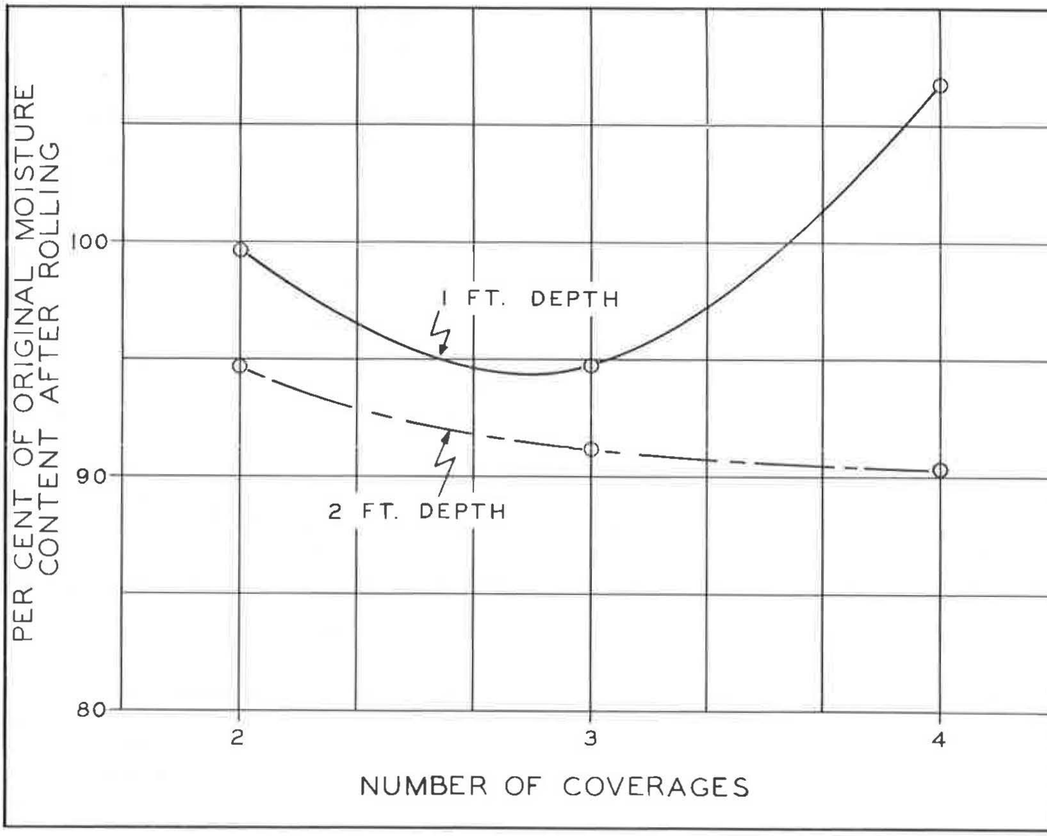

Figure 7. Effects of roller coverages on subgrade moisture contents during rolling.

migrating moisture condition, three coverages appear to be optimum for maximum water removal.

Figure 8 shows the effects of moisture content variation and roller operations on deflection. The moisture content scale is a comparison of the moisture content after rolling to that determined immediately before rolling. The percent deflections are comparisons of the deflections one year after overlaying to those determined before breaking and seating. The operation of the roller in conjunction with the impact hammer gives a relatively consistent change in deflection regardless of any changes in moisture content. This is shown by the horizontal plots of two roller coverages with hammering, and three roller coverages with hammering. Where the roller only was used, changes in moisture content have a definite effect on changes in deflection. However, the most significant indication of this is the effect of three roller coverages and the impact hammer on deflection, regardless of moisture content variation, reducing one-year deflections to less than 10 percent of that before breaking and seating.

A comparison of data on this problem indicates that the use of the impact hammer in conjunction with the roller increases their effectiveness much more than that of either when used for breaking and seating alone.

Figure 9 shows the effects of the number of roller coverages on the deflection at one and two years, expressed as a percentage of the deflection before breaking and seating operations. Of interest is a comparison of the changes in deflection between the use of the roller alone and the use of the roller in conjunction with the hammer. The number of coverages of the roller alone apparently would not require as close a field control as that of the roller and hammer; however, the reduction in deflection would not be of the

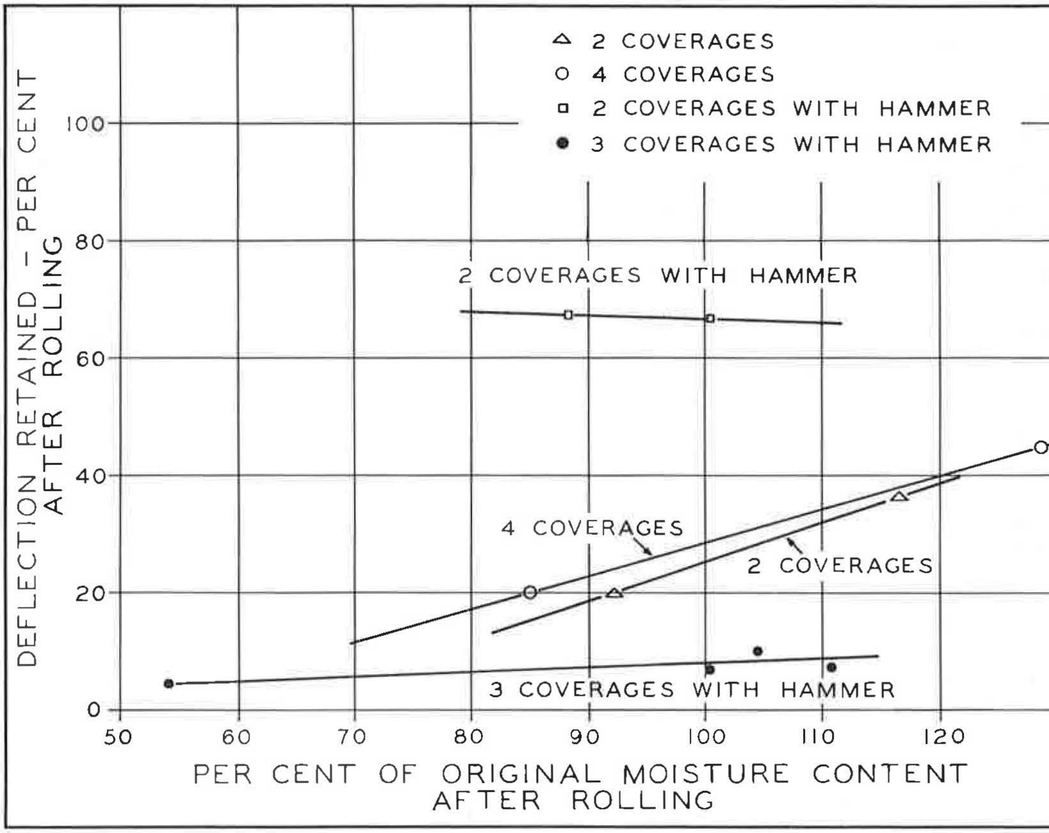

Figure 8. Effects of various roller treatments and moisture content changes during rolling on deflections after 1 yr of traffic vs that immediately before rolling operations.

magnitude as when using the roller and hammer together. Although the number of coverages of the roller, when used with the hammer, appears to be critical, this treatment reduced the deflection, when three coverages were used, more than any other treatment or combination of treatments used in this study. In addition, there is no change in roadway conditions from the one and two years' results where the roller and hammer were used together.

Figure 10 shows the effects roller coverages have on the reflection cracking at one year and at two years after overlaying, as compared to the joints and cracks observed before breaking, seating, and overlaying. Three coverages of the roller with impact hammering eliminated reflection cracking at the applicable test stations. Where the roller only was used, reflection cracking was increased when four roller coverages were applied, indicating overrolling. There is a large amount of change in reflection cracking between the one-year and two-year results where the roller only was used. There is no change in reflection cracking where the hammer and roller were jointly used.

CONCLUSIONS

This study, after two years of service, points out the superiority of a breaking and seating procedure for old pavements on wet subgrades, consisting of a combination of three roller coverages and impact hammering, as shown in Figure 11. The recommended field procedure would be to locate moving slabs with one coverage of the 50-

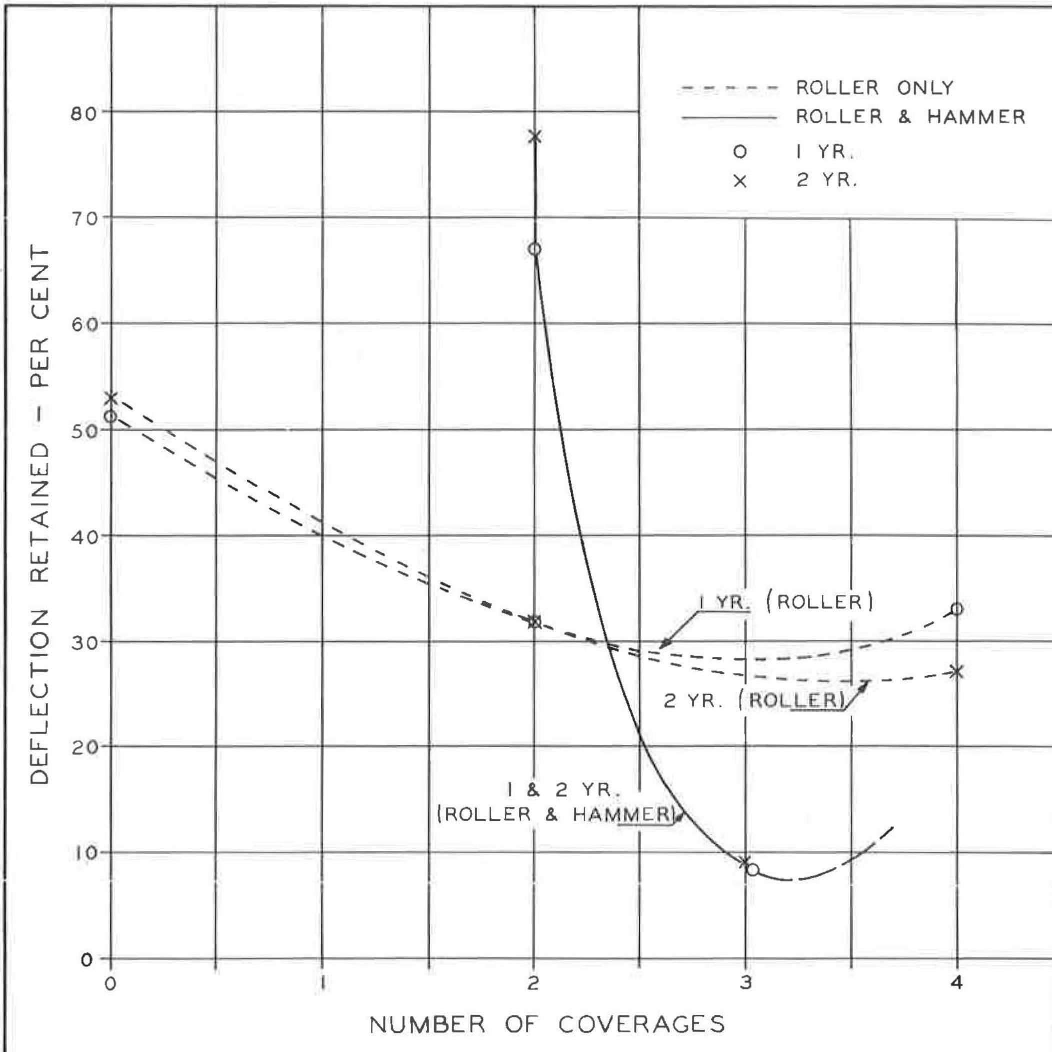

Figure 9. Effects of various roller treatments on Benkelman beam deflections after 1 and 2 yr of traffic vs deflections immediately before breaking and seating operations.

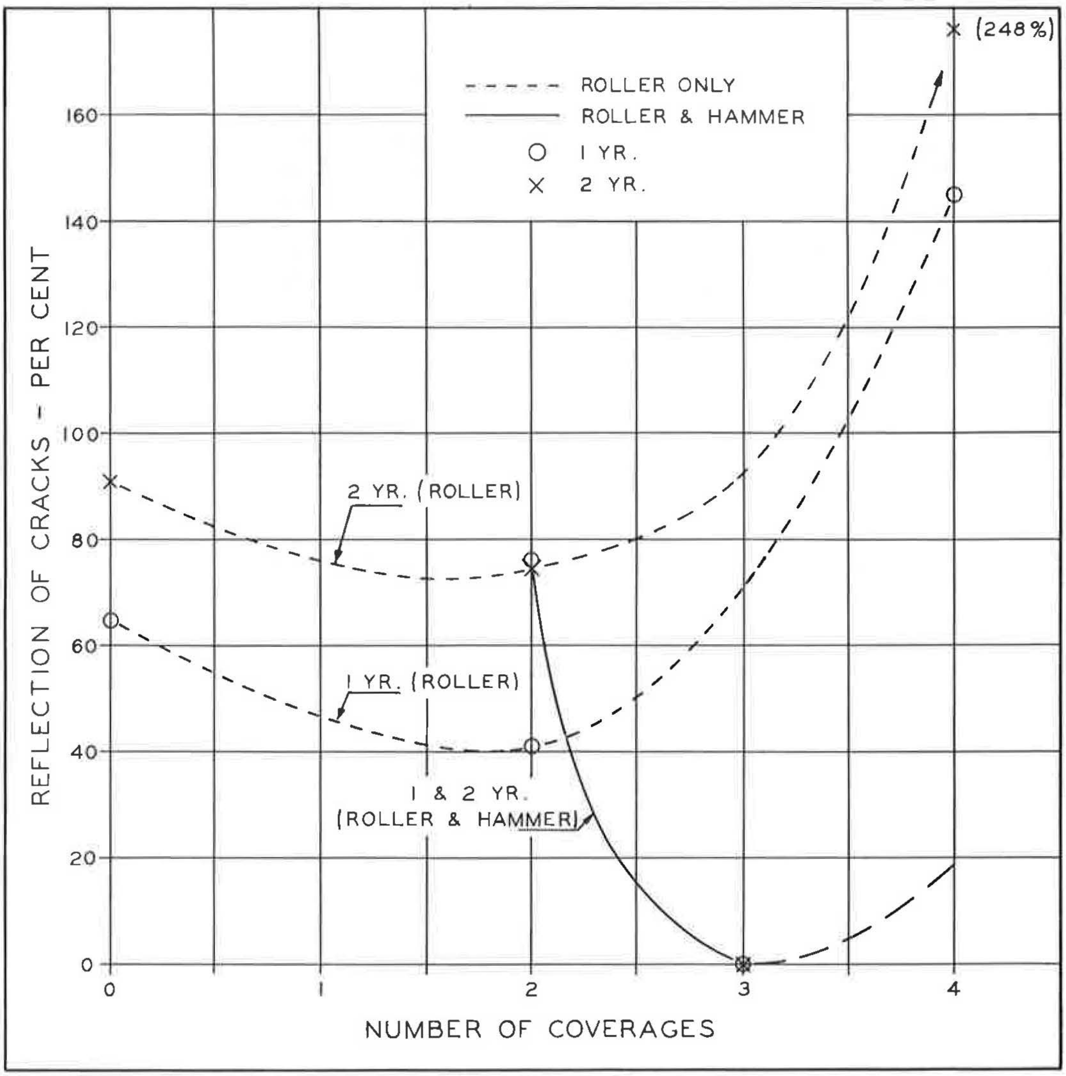

Figure 10. Effects of various roller treatments on reflection cracking after 1 and 2 yr of traffic vs joints and transverse cracks before any treatment.

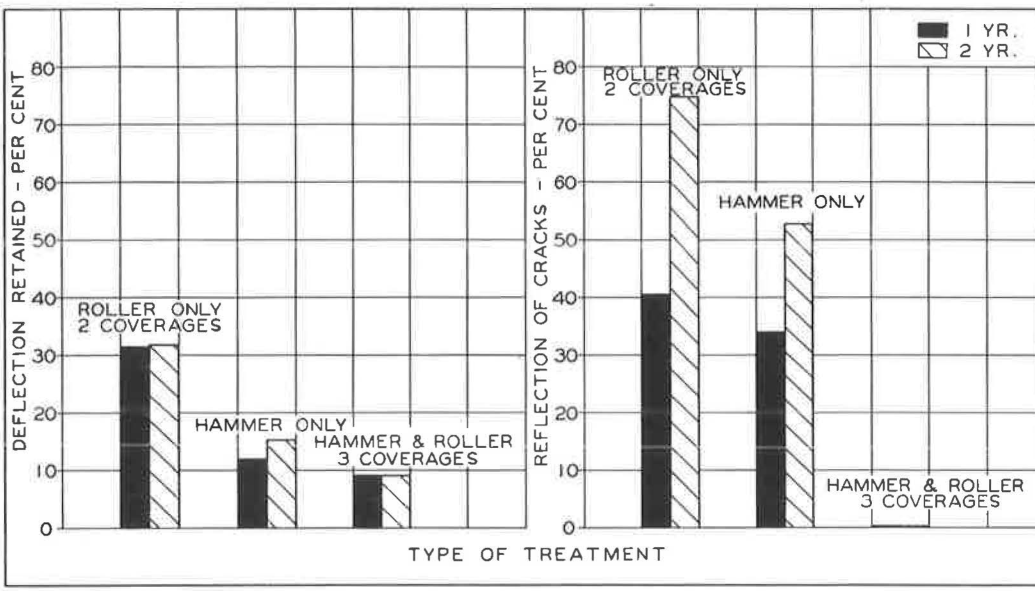

Figure 11. Changes in reflection cracking and deflections for 1 and 2 yr of traffic vs original conditions, for various optimum treatments.

ton roller; to break these slabs with the impact hammer; to apply a seating coverage with the 50-ton roller, also locating additional or continual rocking slabs with this coverage; to accomplish what additional breaking may be required with the impact hammer; and to follow this with the final seating coverage of the 50-ton roller.

Seating Old Pavements with Heavy Pneumatic-Tired Rollers Before Resurfacing

J. L. STACKHOUSE, Maintenance Engineer, Washington Department of Highways

• AFTER REVIEWING the specification requiring the use of a 50- to 60-ton rubber-tired compactor for breaking down panels of old portland cement concrete pavements before resurfacing, it has been concluded that in some cases the compactor is not entirely effective as a means for seating slabs that are bridging over subsided areas in subbases. Instead of specifying a 50- to 60-ton compactor when this type of improvement is contemplated, it has been tentatively decided by the Materials and Construction Divisions that each project should be judged on its own merits. In some cases, the 50-ton compactor is expected to seat the old pavement thoroughly. In other cases, it may take a heavily loaded earth-moving two-axle type of wagon (turn-a-pull type) pulled by a crawler-type tractor and the use of higher speeds to be effective.

During the past two years there have only been two projects using this method because the budget called for constructing highways on new locations and Interstate highway sections. On one project a two-axle, 25-cu yd wagon pulled by a cat D-8 tractor successfully seated the pavement. The vibration of the tractor and the bouncing of the two-axle wagon seated pavement sections that resisted being broken by the compactor unit. A 60-ton compactor was satisfactory for the second.

It is contemplated the unit for breaking the old pavement slabs will be specified and hourly rental rates used as a pay item in the contract. If this is not effective, another type will be negotiated with the contractor.

It has also been concluded that the breaking down of the slabs for seating of an old portland cement concrete pavement is only part of the benefits to be derived from this type of improvement, which is designed to eliminate future subsided areas and prevent reflection cracks coming through the asphalt concrete resurfacing pavement.

The second important part of this type of construction is the placing of a blanket of aggregate over the old pavement with a minimum thickness of 4 in. over the high points. To fill in the deeper subsided areas, 1½-in. minus base course is usually used with a minimum layer of ⅝-in. minus top course. These layers are processed over the old pavement by blading back and forth when in a damp condition and it is believed the fines of the aggregate work down into the cracks of the old pavement and thus securely seat the broken slabs.

This Department has become convinced from an experimental section north of Spokane on PSH 3 that this layer of uncoated aggregate is necessary to prevent reflection cracks from again appearing on the surface. It has also been determined that this layer of untreated aggregate with which the subsided areas of the roadway are filled is much more economical than attempting to level up with a base course of asphaltic concrete. The processing and rolling of the untreated aggregate is beneficial in preventing small subsided areas again appearing for at least a period of five years since these experimental sections have been in use.

After closely inspecting on December 27, 1962, an approximate 5-mi section of highway which was resurfaced by seating the old concrete slabs with a 50- to 60-ton compactor, leveled up with an untreated aggregate and the placement of a wearing course of 3-in. asphaltic concrete, it was found that no reflection cracks have appeared since 1957 or a period of over five years. Inspection was made by walking and/or driving slowly over the pavement when it was in a half-dry, half-damp condition.

The description of this project was reported in Volume 38 of the 1959 Proceedings of

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Paper sponsored by Committee on Salvaging Old Pavements by Resurfacing.

the Highway Research Board. Although no roughometer readings were made on this section, by visual inspection of the surface and as indicated by the centerline stripe there has been very little subsidence of any part of this pavement. The surface is in excellent condition at the present time.