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Agent, Kenneth R., Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, Ky. 40506
Arar, Irfan, Department of Civil Engineering, West Virginia University, Morgantown, W.Va. 26506
Atefi, Ali, Department of Civil Engineering, West Virginia University, Morgantown, W.Va. 26506
Bryden, James E., Engineering Research and Development Bureau, New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, N.Y. 12232
Eck, Ronald W., Department of Civil Engineering, West Virginia University, Morgantown, W.Va. 26506
Graham, Jerry L., Midwest Research Institute, 425 Volker Boulevard, Kansas City, Mo. 64110
Gurney, Gary F., Engineering Research and Development Bureau, New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, N.Y. 12232
Kenyon, Wayne D., Engineering Research and Development Bureau, New York State Department of Transportation, 1220 Washington Avenue, State Campus, Albany, N.Y. 12232
Migletz, James, Midwest Research Institute, 425 Volker Boulevard, Kansas City, Mo. 64110
Pigman, Jerry G., Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, Ky. 40506
Rhudy, Harold C., North Carolina Department of Transportation, 602 Manchester Drive, Raleigh, N.C. 27609; retired
Ritter, James R., Technical and Marketing Services, Potters Industries, Inc., 377 Route 17, Hasbrouck Heights, N.J. 07604
Sack, William A., Department of Civil Engineering, West Virginia University, Morgantown, W.Va. 26506
Usmen, Mumtaz A., Department of Civil Engineering, West Virginia University, Morgantown, W.Va. 26506

Maintenance of Reflective Signs

WAYNE D. KENYON, GARY F. GURNEY, AND JAMES E. BRYDEN

The deterioration of reflective sign sheeting in New York State is examined, and the effectiveness of sign washing and clear-coating for restoring or preserving reflectivity of enclosed-lens engineer-grade sheeting is assessed. Generally, little benefit was achieved by washing all signs in the fall. However, changes in sign brightness, as measured by a retroreflectometer, indicate that significant brightness loss may occur during winter due to dirt accumulation, but that most signs recover after spring rains. The few that do not recover are readily identified by daytime visual inspection and would be the best candidates for washing. A large portion of these chronically dirty signs are located along high-volume highways, close to the pavement, and in industrial areas. The practice of clear-coating did not help maintain sheeting brightness; for the small sample included in this study, it had a detrimental effect. Candidates for replacement can be determined by nighttime visual inspection. Daytime cosmetic appearance alone should not determine need for replacement because signs with cracked or otherwise deteriorated sheeting may still provide adequate night visibility. Average sheeting life could not be determined in this study because no records were available for signs that had been replaced. An examination of the signs still in service determined that service lives of 15 yr in New York State are not unusual for enclosed-lens sheeting. Sign-replacement rates reported by maintenance engineers confirm that engineer-grade sheeting may retain adequate reflectivity for 15 yr or more.

Traffic signs supply information to the motorist; to accomplish this task they must be adequately visible and legible under both day and night conditions. Visibility and legibility requirements have been the subject of considerable research and, by knowing the reflective properties of a sign, it can be predicted whether it will perform adequately to meet driver information needs.

The objectives of this study were to (a) determine the effects on sign sheeting reflectivity of two common maintenance practices--sign washing and clear-coating--and (b) estimate how long sign sheeting retained adequate reflectivity under typical conditions in New York State. In order to accomplish these objectives, a photometer had to be built or purchased that could collect the large amounts of reflectivity data needed.

Highway signs in New York State are constructed of reflective sheeting mounted on a rigid backing (or substrate) and a support structure. When this study began the state required the use of enclosed-lens (engineer-grade) reflective sheeting on highway signs, based on Federal Specification LS-300(A). New York State Department of Transportation standards have since been revised to require encapsulated-lens (high-intensity) sheeting for most signs installed on contract, although the Highway Maintenance Division still uses enclosed-lens sheeting. This study focuses on enclosed-lens sheeting, which represents the majority of signs actually in service around the state.

Signs deteriorate from a variety of causes, and eventually they must be either rehabilitated or replaced. As they age, traffic signs experience irreversible deterioration of the reflective sheeting from the effects of sunlight, weather, airborne abrasives, and air pollution. The sheeting may also experience a reversible dirt buildup from road spray and airborne particulates. This progressive deterioration gradually reduces the visibility and legibility of a sign to the point that a driver may no longer perceive the intended message in time to complete the required response, which results in a reduction in traffic control and safety. Therefore, maintenance practices that retard irreversible deterioration or correct reversible deterioration provide both economic and safety benefits. On the other hand, ineffective procedures not only waste time, but they also divert scarce resources from activities that may provide other benefits.

In 1966 the Highway Maintenance Division implemented a maintenance policy intended to extend the service life of traffic signs. Based largely on the recommendations of manufacturers and the apparent visual effectiveness of washing, this policy required periodic cleaning of all signs. Portable equipment was purchased for each of the 67 residencies in the state to accomplish this task. In the early 1970s an annual sign inspection program was implemented that, by means of nighttime inspections, was intended to identify any signs deficient in terms of legibility and visibility. In addition, research efforts were initiated to investigate specific signing practices and problems.

Hahn, McNaught, and Bryden (1) concluded in 1977 that a substantial proportion of the large guide signs in the state were not adequately legible. Part of this problem was attributable to limited sight distances caused by roadway geometry and overhead structures and to the inherent limitations of the sign materials. However, another part of the problem was also related to the physical deterioration of the sign materials, which had occurred in spite of maintenance activities. Because local priorities varied, as did the availability of personnel and equipment, sign maintenance practices eventually differed among residencies. This variation was further affected by local preferences for sign washing. In addition, some signs were clear-coated in an effort to retard deterioration. This procedure, which involved the application of a liquid coating over the sign face, was recommended by sheeting manufacturers at that time. It also became apparent to Department management that results of the annual inspection program varied from residency to residency. As a result, signs replaced in some counties were in better condition than those left in service elsewhere.

These variations in maintenance practices and sign-replacement policies pointed out the need to establish effective maintenance procedures that could be uniformly implemented in order to direct personnel and materials where they could provide maximum benefits. This study was initiated to determine the effectiveness of sign washing and clear-coating, and to determine how long enclosed-lens sheeting retained adequate reflectivity under typical New York State conditions.

It is recognized that signs may fail for reasons other than loss of sheeting reflectivity. However, problems such as vandalism and accident damage are site dependent and generally not related to properties of the sign material. Although information on these causes of failure may be important to the choice of sign materials, they are beyond the scope of this study.

INVESTIGATION

Deterioration Types

In order to address the objectives of this study it is necessary first to understand the deterioration modes experienced by sign sheeting. Drawing on information supplied by maintenance personnel, sign fabricators, and sheeting manufacturers, five types of sheeting deterioration were identified.

These types can best be understood after examining the physical makeup of enclosed-lens sheeting. This material consists of a layer of transparent plastic of the appropriate color in which glass beads are embedded. A metallic reflector shield is provided behind the plastic, plus a layer of adhesive and a protective liner that is removed during sign fabrication. The deterioration types normally encountered are as follows.

1. Clouding and color fading: Exposure to ultraviolet rays in sunlight and atmospheric pollutants may result in gradual clouding and deterioration of the transparent plastic and metallic reflector layers. This results in diminished color contrast between legend and background and may cause loss of reflectivity.
2. Cracking: Differences in thermal expansion between the sheeting and metal substrate are believed responsible for cracking. Brightness is not directly affected, but daytime appearance may suffer.
3. Abrasion: Microscopic surface deterioration, which produces a rough feeling on the originally smooth surface, may be caused by windborne particles or chemical corrosion. Brightness is adversely affected and daytime condition may appear dull because the plastic layer is less transparent.
4. Delamination: Full-depth separation of the sheeting from the substrate may result from insufficient adhesion caused by either manufacturing or fabrication problems. Internal layer separation within the sheeting may result from manufacturing problems. Either distress mode diminishes both brightness and daytime appearance.
5. Dirt accumulation: The accumulation of dirt on the sign is the most common type of deterioration; it affects most signs at various periods during their service life. Unless the dirt becomes deeply embedded, it can be removed by washing--either by natural rainfall or by maintenance activity--and the brightness and appearance of the sign can be restored.

Most older signs in New York exhibit the first four distress modes in varying degrees of severity, and all signs exhibit dirt accumulation. Frequently other distress modes, which may not be immediately recognizable, can be explained by a combination of these general types. For example, a deeply cracked or abraded surface may permit the underlying dark adhesive layer to bleed through and spread from the scar, thereby resulting in the appearance of metal corrosion on the surface of the sheeting.

The maintenance activities examined in this study address two of these five problems. Clear-coating was intended to reduce and restore the effects of minor abrasion, and washing removes dirt accumulation.

Rating Sign Condition

Both color and brightness contrast are important parameters that affect sign visibility and legibility. Under diffuse illumination--daylight or artificial nighttime illumination--sign brightness is generally adequate to permit detection. Color contrast provides much of the legend-background distinction required for legibility. Legends mounted on backgrounds with similar brightness but different colors are legible, but similarly colored legends and backgrounds with greatly differing brightness are much less legible.

At night under headlight illumination the observation angle between the light source and the driver's eye is small, and sign detection becomes dependent on the retroreflective properties of the background and legend. For detection, brightness contrast between the sign and its surroundings is most important; for legibility, brightness contrast between the legend and background is also important, playing an even greater role than color contrast.

Based on discussions with maintenance personnel and the examination of discarded signs, it is apparent that loss of reflectivity, as identified by the nighttime surveys, is the principal reason for sign replacement. Although funding limitations have reduced sign replacement to a general low level statewide, establishing priorities for replacement varies among residencies, which reflects the subjective nature of nighttime surveys. Although this variability is not usually great in terms of the condition of signs actually replaced, it does point out the need for an objective measure of sheeting reflectivity for use in this study to permit uniform and consistent evaluation of sign condition in the field.

Originally, it was considered desirable to develop a prototype instrument in-house for use in this project that could then be reproduced inexpensively for each of the 10 regional offices of the Department. This would permit uniform evaluation of sign-replacement needs around the state. Unfortunately, staff reductions early in this project made it impossible to accomplish this goal.

Instead, a commercial retroreflectometer--the Gamma Scientific Model 910B--was purchased for this project. This device consists of an internal light source and photocell. Target distance and incidence and divergence angles are controlled by the geometry of the equipment. Calibration is provided through the use of small discs of reflective sheeting in each standard color that were premeasured in the laboratory to establish their specific intensity. An evaluation performed by the Pennsylvania Department of Transportation (2) confirmed that the specific intensity of sign sheeting--candelas of reflected light per footcandle of incidence light per square foot of target ($cd/ft\text{-}c/ft^2$)--can be quickly and reliably measured by using this device, thus eliminating bias due to ambient light, viewing position, and individual rater. Although this instrument proved suitable for use in this project, it is too costly and time consuming for use in annual sign condition surveys.

Experimental Design

This study was designed to examine the effects of two maintenance activities: sign washing and clear-coating. In addition, an effort was made to relate reflectivity to sign age to determine the service life provided by enclosed-lens sheeting.

Two sample groups were selected to determine the effects of sign washing. The first group, which was located in the Albany area, was chosen to represent the general statewide sign population. A total of 153 signs were selected: 67 with white backgrounds, 53 with yellow, and 33 with green. Highway types included urban and rural expressways, urban arterials, and suburban and rural primary and secondary routes. Sign locations varied from immediately off the pavement to more than 20 ft from the pavement edge.

A second sample was selected in highly industrialized areas in and around Buffalo and Syracuse to represent severe environments in terms of dirt accumulation. This sample included 28 signs with white backgrounds, 17 with yellow, and 15 with green. Roadway types included urban and suburban expressways, arterials, and primary routes. Sign locations were similar to the Albany sample. For simplicity, these groups are referred to as the non-industrial and industrial samples.

Physical and historical data were recorded for each sign, including installation date (when available). Although these samples included a wide range of signs of different ages and conditions, badly deteriorated signs (those that were peeling, faded, or nonreflective) were not included because it was not considered worthwhile to continue maintaining such signs.

The reflectivity of each sample was measured four times: in the fall before and after washing, and in the winter and spring to determine the effects of the dirt reaccumulation. The Albany sample was measured during the winter of 1977-1978 and the Buffalo-Syracuse sample during the winter of 1979-1980. Washing was accomplished by a two-person field crew by using a portable pump, water tank, and mild industrial detergent. Reflectivity of each sign was first measured in its existing condition in the fall. Loose debris was then hosed off the sign surface, and it was scrubbed down by using a soft-bristle brush, taking care not to abrade the surface. The sign was then thoroughly rinsed and allowed to dry for a few hours to a few days before reflectivity was measured again. Winter measurements were made during a period when dirt accumulation would be near its peak from winter road spray combined with lack of rainfall during winter months; the spring survey was made after several heavy rainfalls had provided natural washing.

Reflectivity was measured only on the background sheeting. For white and yellow backgrounds, the nonreflective black legends provide negative contrast, and it is necessary only to measure background reflectivity. Most green signs included white button-reflector legends; although it would have been desirable to measure legend reflectivity, this was not possible with the instrument used. Thus the effect of dirt accumulation on the background was also assumed to represent legend condition.

Depending on the size of the sign, five replicate reflectivity measurements were made at four to nine locations across the sign face to obtain a fair representation of sign reflectivity. Generally, variability within the sign was not great. Five reflectivity measurements were deemed sufficient, and survey time was held to a reasonable level. This was especially important on larger signs, where ladders or scaffolds had to be moved between measurements.

The sign-installation dates were combined with measured reflectivity to provide a measure of sheeting life. However, this method has two built-in biases that may have failed to consider signs with shorter-than-normal service lives. First, a few badly deteriorated signs still in service were not included in the sample because they were no longer reflective and thus were not expected to benefit from washing. Second, signs that had been removed from service because of poor reflectivity, vandalism, accident damage, or other causes were not included. Thus, although the signs included in the survey indicate the range of service lives possible, they do not provide the overall distribution of service lives to be expected.

The survey sample taken to examine the effects of clear-coating was limited because of personnel limitations within the research staff and a general scarcity of clear-coated signs around the state with complete records to determine when they were coated. Nevertheless, one section of highway located in the Mohawk Valley west of Albany included a number of signs with green or white backgrounds installed in 1972 and clear-coated in 1973. In addition, one green sign had clear-coating applied to half its surface. Signs of similar age without clear-coating were located on the same or adjacent highways to compare with clear-coated signs. In all, five clear-coated green signs and six clear-coated white signs were compared to five non-clear-coated signs in each color, plus the additional half-treated sign.

Survey of Maintenance Practice

Sign inspection, repair, and replacement are performed by personnel from the Department's residencies. Telephone interviews were conducted with the resident engineer or sign foreman at each residency to determine current sign maintenance practices, special problems, and suggested improvements. Follow-up visits were made to three residencies to gather more specific information. The survey information collected was used in assessing data collected in the other phases of this project and in obtaining candidate signs and data on the ages of the signs.

RESULTS

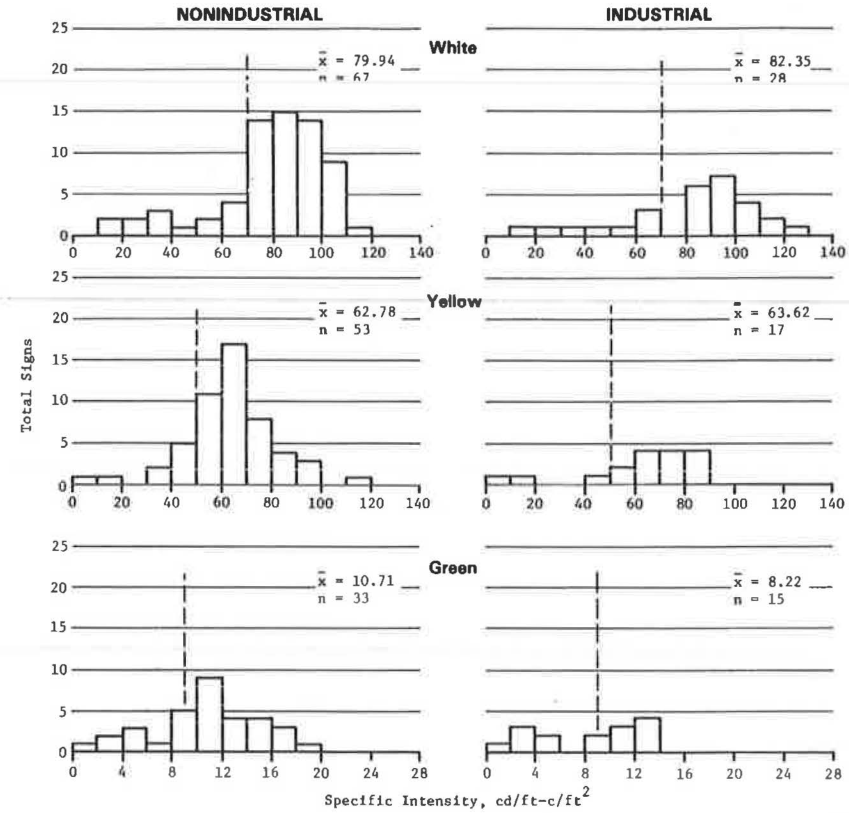
Effects of Brightness on Sign Performance

Distributions of average before-washing specific intensities measured for the 213 signs included in the two field samples are shown in Figure 1. Also shown are the minimum specification levels for new sheeting; the data indicate that less than 20 percent of the nonindustrial and 30 percent of the industrial signs were less than the minimum specification levels--most by only a small amount. Nevertheless, χ^2 test values indicate that the proportions of signs less than the specification level are not significantly different for the two samples for any of the three colors.

In order to evaluate the effects of washing and clear-coating, the effects of background brightness on sign detection and legibility must be understood. Other research on sign conspicuity (3) indicates that a large increase in brightness is needed to provide a noticeable increase in detection distance. For example, changing from enclosed-lens to encapsulated-lens sheeting, which is about a threefold increase in brightness, improves detection distance by only 20 percent under the best conditions. Even larger increases in background brightness have little effect on legibility. One recent report (4) also indicates that substantial decreases in background brightness less than the specified levels of enclosed-lens sheeting in most cases detract only slightly from legibility. For white and yellow engineer-grade sheeting with black negative-contrast legends, a specific intensity of 30 cd/ft-c/ft^2 provides almost the same legibility distance as sheeting at the specification level: 70 cd/ft-c/ft^2 for white and 50 cd/ft-c/ft^2 for yellow. For green background sheeting with encapsulated-lens or button-copy legends, a drop in background reflectivity to 5 cd/ft-c/ft^2 also provides nearly the same legibility as the minimum specification level of 9 cd/ft-c/ft^2. Further examination of the data in that report indicates that a decrease in specific intensity of 15 percent of the minimum specification level is about the smallest change that would be detectable under any conditions, even though substantially larger losses have little practical effect.

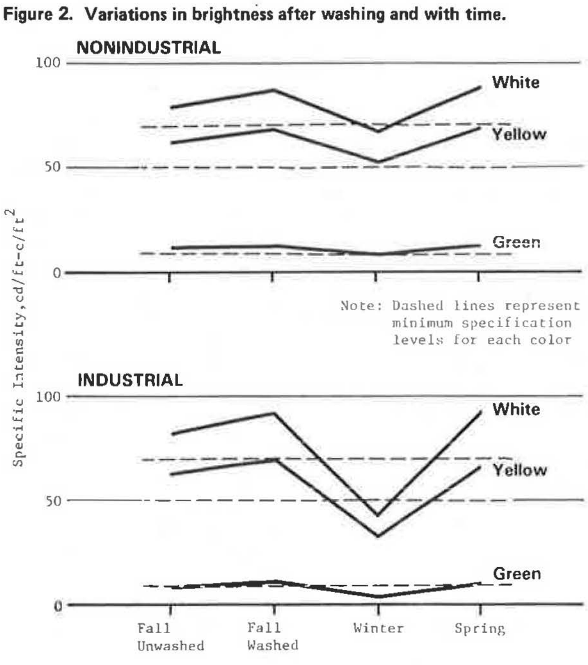
Because small losses in brightness due to dirt accumulation and abrasion may be detectable under some conditions and may be accumulative, thus indicating further loss of brightness with time, a brightness loss of 15 percent is considered in this analysis to be the lower limit of practical signifi-

Figure 1. Distribution of brightness measurements before washing.



Note: Vertical dashed lines are minimum specified specific intensities for new enclosed-lens sheeting.

Figure 2. Variations in brightness after washing and with time.



cance. Only when brightness loss exceeds this value is there any concern that sign performance may eventually suffer.

Effects of Sign Washing and Seasonal Changes on Dirt Accumulation

Changes in sign reflectivity after washing and with time are shown in Figure 2, which also shows average brightness for both industrial and nonindustrial samples in all three colors. Fall sign washing produced a small average increase in brightness, ranging from 7 to 11 percent for these six groups. Dirt accumulated again during the winter, and the average brightness losses ranged from 24 to 53 percent of the washed values. Following the spring rains most of the group averages returned approximately to the washed averages. By using the data in this figure, it appears that the industrial-area signs were subject to greater changes than the signs in nonindustrial areas. Although average sign brightness followed predictable trends, individual signs were subject to considerable variability from the group averages, and dirt accumulation was great enough to cause loss of legibility in some cases.

Although most signs improved somewhat after washing, a few appeared to have less reflectivity after washing, but this is attributed to instrument and measurement variability on already-clean signs. Few of the washed signs experienced brightness changes greater than the 15 percent level considered detectable to a human observer, and only one--a white, industrial-area sign--improved sufficiently to have a large impact on sign performance.

Most signs became dirtier during winter to the extent that brightness loss would be noticeable, and some were so dirty that sign performance would not be adequate. All signs that suffered reduced legibility because of dirt accumulation were readily apparent from simple daytime visual inspection. Those that did not appear visibly dirty did not experience significant brightness loss from the washed condition.

Following the spring rains, the average sign brightness returned to approximately the washed fall values. Nevertheless, there is again considerable variability within the samples, which indicates that individual signs are affected by their particular environments. Part of this variability is also attributable to instrument and measurement variability.

The data in Table 1 summarize the number of signs that experienced noticeable brightness changes (i.e., exceeding 15 percent of the minimum specification level) after washing and during winter, and also the number of signs that failed to recover within 15 percent of the washed value in the spring. The apparent differences between the nonindustrial and industrial groups were examined by using the χ^2 test. After washing, the proportion of signs that improved more than 15 percent was about the same for each sample--25 and 32 percent, respectively.

In winter a significantly larger proportion of industrial signs became noticeably dirty--90 percent compared with 65 percent for the nonindustrial group. Similarly, following spring rains a significantly larger proportion of industrial signs failed to return to within 15 percent of the washed values--27 percent compared with only 7 percent for the nonindustrial signs.

It has been shown that, with the exception of the winter period, most signs receive little benefit from washing because natural cleansing from rainfall achieves nearly the same effect. Nevertheless, it is also clear that some signs do benefit from washing. The data in Table 1 indicate the proportions of signs that benefit from washing; those proportions are not significantly different from the industrial and nonindustrial samples.

The data in Table 2 examine the other two environmental parameters that may be expected to affect dirt accumulation: traffic volume and distance from the pavement. The χ^2 test was again used to determine if signs close to higher-volume roads [i.e., within 11 ft of roads with annual average daily traffic (AADT) greater than 21,000 vehicles] responded more to washing and dirt accumulation than signs farther from the road or on lower-volume roads. After initial washing, this group did not respond significantly more than the remainder of the sample. During winter the signs at these locations from the nonindustrial sample experienced significantly more dirt accumulation than the rest of the sample: 83 percent had brightness loss of 15 percent or more compared with only 54 percent for the rest of the sample. Considering industrial signs, nearly the entire population experienced brightness losses greater than 15 percent. In spring most signs recovered to within 15 percent of the washed value. However, for both the nonindustrial and industrial samples, the proportion failing to recover was higher in the critical locations--close to high-volume roads.

It appears paradoxical that this critical category did not respond more to washing than the rest of the signs, considering that more of them failed to recover in the spring. However, the washed readings were taken in the fall, and it is possible that the additional 6-month exposure to rain would result in recovery of most of these signs still experiencing detectable dirt accumulation in the spring.

Further examination of the industrial sample revealed that a larger portion of these signs were close to high-volume roads: 25 of 60 signs compared with only 23 of 153 signs in the nonindustrial sample. This bias was unavoidable because roadways in the industrial areas where the signs were selected generally had high traffic volumes compared with the nonindustrial sample. The analysis so far has indicated that industrial signs became dirtier in winter and recovered less in spring. Further, it was revealed that winter dirt accumulation and failure to recover in spring were more critical for signs close to high-volume roads. The question then arises whether apparent differences between the two samples may be attributable to the higher portion of industrial signs located close to high-volume roads.

Table 1. Changes in brightness caused by dirt accumulation and washing.

	Signs			
		Exceeding 15 Percent Change		
Sign Group	Total	After Washing[a]	Winter[b]	Spring[c]
Nonindustrial				
White	67	14	43	5
Yellow	53	13	31	1
Green	33	11	15	5
Total	153	38	89	11
Industrial				
White	28	9	27	6
Yellow	17	5	15	3
Green	15	5	12	7
Total	60	19	54	16
All signs	213	57	143	27

[a]Washed-unwashed exceeds 15 percent of minimum specification value.
[b]Washed-winter exceeds 15 percent of minimum specification value.
[c]Washed-spring exceeds 15 percent of minimum specification value.

Table 2. Effects of traffic volume and distance from pavement on dirt accumulation.

	No. of Signs Affected by Distance from Pavement and Traffic Volume				
	≤11 ft		>11 ft		
Variable	≤21,000 AADT	>21,000 AADT	≤21,000 AADT	>21,000 AADT	Total
Total signs					
Nonindustrial	38	23	50	42	153
Industrial	8	25	8	19	60
Signs with change greater than 15 percent					
Washed-unwashed					
Nonindustrial	3	5	13	17	38
Industrial	3	11	0	5	19
Washed-winter					
Nonindustrial	10	19	29	31	89
Industrial	7	25	6	16	54
Washed-spring					
Nonindustrial	0	7	2	2	11
Industrial	1	12	0	3	16

That question is examined in Table 3, where each sample is stratified according to proximity to high-volume roads. After washing, no significant change is apparent between the industrial and nonindustrial samples for either subgroup. In winter the industrial sample close to high-volume roads fared slightly worse than the nonindustrial sample, but the proportions are too small to be examined by using the χ^2 test. Nevertheless, for those signs not close to high-volume roads, significantly more of the industrial signs experienced noticeable dirt accumulation than did the nonindustrial group: 83 versus 54 percent. Finally, spring recovery was slightly less for industrial signs in both subgroups, but differences between the two samples are not significant. Thus it appears that signs in industrial areas do experience somewhat more dirt accumulation that cannot be attributed solely to their location close to high-volume roads.

Clear-Coating

Before 1978 a major manufacturer of enclosed-lens sheeting recommended that signs be clear-coated in the field after approximately 4 yr of service. For signs exposed to road salts in northern climates, initial clear-coating was also recommended (5). However, clear-coating is no longer recommended in that company's literature. Further information on clear-coating was obtained from the survey of maintenance personnel, which determined that clear-coating has been virtually eliminated as a maintenance procedure. Most respondents considered clear-coating to be of little value, and some even indicated that it had damaged signs. Even when successful, adherence to strict application procedures was necessary in order to obtain satisfactory results. An earlier study by the Engineering Research and Development Bureau found that overspray from clear-coating seriously reduced the reflectivity of button copy (6).

One residency had been a strong advocate of clear-coating to prolong sign life and had maintained records of clear-coated signs, which permitted an examination of the effectiveness of this procedure. Route markers and guide signs installed along one route in 1972 were clear-coated in 1973. The field clear-coating applied after 1 yr was an attempt to obtain the same protection achieved with the initial clear-coating recommended by the manufacturer. These signs were inspected by research personnel in 1981, and reflectivity was measured on a sample of several signs. Observations and measurements were also made on a sample of signs of similar age on adjoining routes that had not been clear-coated.

Except for some slight surface abrasion, the clear-coated signs appeared to be in satisfactory condition. Nighttime inspection confirmed satisfactory visibility and legibility. Nevertheless, comparison with signs without clear-coating (Table 4) indicates that the latter remain much brighter. Green-background guide signs without clear-coating, ranging in age from 7 to 9 yr, were about 50 percent brighter than the 9-yr-old clear-coated signs. The white route-marker signs revealed an even greater difference: the 7-yr-old signs without clear-coating were, on average, more than 3 times brighter than the 9-yr-old clear-coated signs.

As noted earlier, along this same route one guide sign had clear-coating applied to half the surface while the other half was left uncoated. After 9 yr--8 yr after clear-coating--the coated portion was scaling while the uncoated side remained in good condition. Reflectivity measurements indicated that the uncoated side was 65 percent brighter than the coated portion.

In summary, manufacturers' recommendations and maintenance practice no longer include clear-coating as a standard procedure. Reflectivity measurements on a limited number of signs clear-coated after 1 yr

Table 3. Comparison of industrial and nonindustrial signs close to high-volume roads.

Condition	Signs with Brightness Change <15 Percent	Signs with Brightness Change >15 Percent	χ^2
Signs Close to High-Volume Roads[a]			
After washing			
Nonindustrial	18	5	-
Industrial	14	11	1.76
Winter			
Nonindustrial	4	19	-
Industrial	0	25	-
Spring			
Nonindustrial	16	7	-
Industrial	13	12	0.89
Signs not Close to High-Volume Roads[b]			
After washing			
Nonindustrial	97	33	-
Industrial	27	8	0.007
Winter			
Nonindustrial	60	70	-
Industrial	6	29	8.50[c]
Spring			
Nonindustrial	126	4	-
Industrial	31	4	-

[a] <11 ft and >21,000 AADT.
[b] >11 ft or <21,000 AADT.
[c] χ^2 exceeds 95 percent confidence level.

Table 4. Comparison of reflectivity of clear-coated and uncoated signs.

White Route Markers: Coated Avg Brightness (cd/ft-c/ft)	Coated Age (yr)	Uncoated Avg Brightness (cd/ft-c/ft)	Uncoated Age (yr)	Green Guide Signs: Coated Avg Brightness (cd/ft-c/ft)	Coated Age (yr)	Uncoated Avg Brightness (cd/ft-c/ft)	Uncoated Age (yr)	Partly Coated Green Guide Sign: Brightness Measurement (cd/ft-c/ft) Coated	Uncoated
38.3	9	89.1	7	9.48	9	16.55	7	6.4	12.0
29.0	9	85.8	7	13.04	9	12.40	9	4.8	12.2
22.0	9	90.1	7	11.87	9	14.66	7	8.2	13.2
14.9	9	87.2	7	9.27	9	16.30	9.25	10.6	12.2
26.0	9	87.5	7	7.50	9	16.00	9.25		
36.0	9							7.5[a]	12.4[a]
		87.9[a]	7[a]	10.23[a]	9[a]	15.18[a]	8.3[a]		
27.7[a]	9[a]								

[a] Sample average.

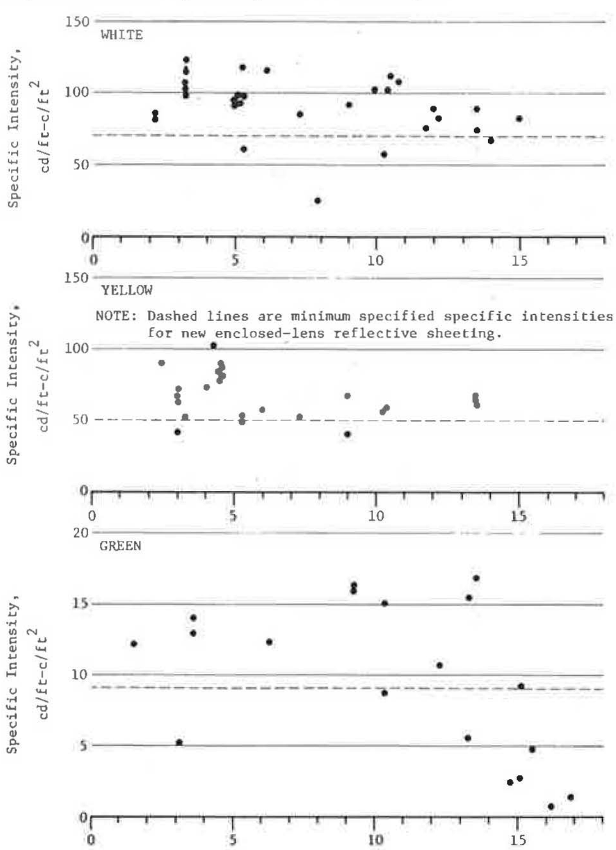
of service indicate that this procedure does not preserve sign reflectivity and may actually cause a more rapid drop in brightness.

Sheeting Service Life

The age and average brightness of 63 signs in the nonindustrial sample are shown in Figure 3. These are the only signs for which accurate age data could be obtained from maintenance records. Based on this sample it appears that almost none of the white or yellow signs have experienced substantial losses in reflectivity after as long as 15 yr in service, and the green signs generally remain in satisfactory condition for at least 13 yr. These data do not indicate a pronounced brightness loss with age, and it is obvious that some signs will provide service lives of 15 yr or more. Nevertheless, it should not be concluded from this sample that average sheeting life will reach this figure. It must be remembered that, of the signs still in service, only those in satisfactory condition were included in the original sample. In addition, signs already replaced because of unsatisfactory sheeting condition are not included. Some of these discarded signs were examined, and it was apparent that they were removed from service for various reasons, including loss of reflectivity, cracking, and vandalism. However, the installation date was included on only a few signs, and the dates when the signs were taken out of service were not provided in any cases. Thus these discarded signs confirmed that replacement occurs for a variety of reasons, but they did not offer any usable information relating to length of service provided.

Interviews with maintenance personnel also provided information about sheeting life. The average

Figure 3. Effects of age on brightness of washed signs.



life was estimated to be between 5 and 13 yr, although extreme estimates ranged from 3 to 20 yr. However, the statewide average replacement rate for unsatisfactory reflectivity was only 3 percent of the total annual sign population. The respondents pointed out that the life of any individual sign may vary considerably from the average. Factors that affect sign life include roadway type, sign location relative to the pavement, sign substrate, and local air quality. The direction a sign faces determines its exposure to ultraviolet radiation as well as to windblown abrasives. Nevertheless, for the 63 signs included in Figure 3, no relation between sign direction and reflectivity was apparent. Some signs facing in each direction are included among those still in satisfactory condition.

In order to obtain an unbiased estimate of sheeting service life, it would be necessary to start with a sample of new signs and follow the history of each to failure. However, even such an effort might be unproductive because of manufacturing changes that result in changes in the actual sheeting installed over time. Thus, by the time average sheeting life is reliably estimated, the material supplied may differ considerably from that tested.

It is impossible to estimate average life reliably for enclosed-lens sheeting based on these data, but based on experience of maintenance personnel around the state and on the long life of sheeting measured in this evaluation, the manufacturer's estimated life of 7 yr appears to be conservative for many of the signs in the state.

Interviews with maintenance personnel and night inspection visits confirmed that signs with deficient brightness are readily apparent at night. Thus, although this method is not precise, it is adequate for programming needed sign replacements. The interviews and field visits also indicated that signs occasionally are replaced because of severe cracking. For example, the badly cracked sign shown in Figure 4 has a specific intensity of 94.2 cd/ft-c/ft^2, and it still provides satisfactory nighttime visibility and legibility. Thus replacing

Figure 4. Badly cracked sign, but brightness is still satisfactory, with specific intensity of 94.2 cd/ft-c/ft^2.

signs based on daytime inspection alone may result in decisions based on aesthetics rather than night performance.

SUMMARY AND FINDINGS

Most of the 213 signs measured in a fall survey had background brightness close to or above the minimum specified levels for new white, yellow, and green engineering-grade reflective sheeting. Although most signs experienced a small improvement in brightness after washing, most improvements were too small to be of any practical significance in terms of legibility or visibility; i.e., less than 15 percent of the minimum specified brightness. During winter months dirt accumulation reduced the brightness of most signs, and some became so dirty that legibility was adversely affected. Nevertheless, spring rains restored nearly all the signs close to their washed brightness. Other than during winter months, when dirt accumulation may be severe in some cases, most signs appear to be kept sufficiently clean by natural washing, and little added benefit is achieved by washing in terms of legibility or visibility. Those few signs that would benefit from washing should be readily apparent from daytime inspection because of severe dirt accumulation on the sign face.

Signs close to high-volume roads and signs in industrial areas were subject to greater dirt accumulation during winter months, but most recovered their brightness after spring rains.

Clear-coating is no longer recommended as a standard maintenance procedure. Brightness from a limited sample of signs 7 to 9 yr old that were clear-coated after 1 yr of service indicated that this procedure did not help maintain sheeting brightness; it actually resulted in lower brightness values than sheeting that was not clear-coated.

Sufficient data could not be obtained to determine the average life or overall range of service lives provided by engineering-grade sheeting. Few signs included in this survey had brightness values below the minimum specification levels at ages up to 15 yr, but the study sample did not consider signs removed from service because of sheeting failure that may have occurred at an early age. Replacing signs based on daytime appearance alone may result in unnecessary replacement. Nighttime inspections, however, appear to provide ready identification of signs with deficient reflectivity.

Based on the results of this study, the following findings can be stated.

1. A fall survey of 213 highway signs constructed with engineering-grade reflective sheeting revealed that most had brightness values near or above minimum specification levels for new sheeting. However, this sample contained only signs that still appeared to be in serviceable condition.
2. Little benefit in terms of improved brightness was achieved by washing the signs in the fall.
3. Most signs experienced significant dirt accumulation during winter months, some to the point that legibility became inadequate for a period of time.
4. Signs located close to high-volume roads and in industrial areas experienced greater dirt accumulation during the winter.
5. Spring rains restored most signs to within 15 percent of their washed-fall brightness.
6. Signs that may benefit from washing, and then generally only during winter months, can be identified by daytime inspection.
7. Clear-coating after 1 yr in service did not maintain sheeting brightness. Instead, it had a detrimental effect on the small sample included in this study.
8. Average service life could not be determined in this study, but engineering-grade sheeting appears to be capable of providing at least 15 yr of service life in upstate New York.

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New-Old Cost-Cutting Concept in Traffic Marking

HAROLD C. RHUDY AND JAMES R. RITTER

Paint is the predominant traffic marking material in use today. With respect to traffic marking, the needs of traffic engineers, as expressed in 1950, are similar to the needs of traffic engineers today. North Carolina has used a premixed paint dual system for more than 30 years, and has gained experience about the benefits of premixed paint and has solved many of the equipment problems. The experience in North Carolina has indicated that, when compared with a standard paint line, the premixed dual system has the following characteristics: increased service life, extended night visibility, a 3-min dry time, satisfactory daytime visibility, and lower cost. The different theories about a standard line and a premixed line are described. A white centerline placed as a premixed dual line in 1970 is still effective after 12 years of use. Of the 46,000 paved miles of roads in North Carolina, the premixed system is so durable that less than 1 percent of the roads need to be restriped during the same season. The cost savings to the state are substantial, and a low cost per foot of applied line per day of useful life is obtained. The questions raised concerning the use of premixed paint are discussed, and some of the misconceptions held about the premixed dual system are explained. Suggestions are also given on how to specify striping equipment for applying premixed paints.

The greater mobility of people has continually applied pressure on traffic engineers to upgrade their traffic marking systems. In 1950 the relative order of preference for future traffic marking systems, as emphasized by state highway departments, was as follows (1):

1. Increased service life,
2. Night visibility,
3. Increased rate of drying,
4. Storage stability,
5. Day visibility, and
6. Lower cost per gallon.

The needs of state highway departments in 1950--obtaining a fast drying pavement marking material that would have increased durability for day and night visibility and a low cost per foot of applied line per day of useful life--are similar to current needs. Also, the predominant striping material used in 1950 and in use today is traffic paint.

Therefore, it is appropriate to either introduce or reintroduce a new-old cost-cutting concept in the use of traffic paint. The traffic marking system used in North Carolina has saved millions of dollars and many lives over more than 30 years of use. This method is a dual system of premixed traffic paint that has glass spheres dropped on the surface.

BACKGROUND

The premixed dual system was available and was used before 1950. It was reported in 1950 (1) that 50 percent of the states were using a premixed system and 33 percent of the 48 states were using a premixed dual system. In 1955 the states that used a premixed system dropped to 40 percent, and only 17 percent used a premixed dual system (2). In 1965 only 20 percent of the states were using a premixed system of any type (3).

As states decreased their use of the premixed dual system, North Carolina increased the amount of premixed dual system that was applied. For example, in 1950 North Carolina used 95,600 gal of premixed traffic paint and 573,000 lb of glass spheres. By 1960 use had increased to 315,800 gal and 1,894,800 lb; and in 1965 it had increased further to 767,515 gal and 4,605,000 lb. During fiscal year 1981-1982, North Carolina used 840,000 gal of premixed traffic paint and 5,100,000 lb of glass spheres. During the period between 1950 and 1982, approximately 58 percent of the glass spheres were premixed in this paint and 42 percent were drop-on glass spheres.

PREMIXED DUAL PAINT SYSTEM

There are two basic traffic paints available for general highway use. One is a traffic paint in which glass spheres are dropped onto the wet paint immediately after application; this is called the drop-on paint system. The other system adds glass spheres to the paint during the manufacturing process, and this mixture is then sprayed on the road. Additional glass spheres are dropped onto the wet paint film during the application, thereby giving immediate retroreflectivity; this is called a premixed dual (or combination) paint system. Only these two systems are discussed in this paper (plain paint and premixed paint without drop-on glass spheres are not acceptable for general highway use).

A normal plain traffic paint has approximately 6 lb of glass spheres per gallon dropped onto the surface of the wet paint line. The North Carolina premixed traffic paint has a minimum of 3.5 lb of glass spheres per gallon added during the manufacturing process. An additional minimum of 2.5 lb of drop-on glass spheres per gallon are dropped on the wet paint for immediate nighttime reflectivity. Therefore, the 6 lb of glass spheres per gallon is equal in both systems.

In order to appreciate the differences in a standard drop-on line and a premixed system, it is useful to understand both systems. The drop-on glass spheres used by North Carolina are similar to those used by other states (4). The sizes of the glass spheres are given in the following table:

U.S. Standard Sieve Size	Percentage Retained Minimum	Percentage Retained Maximum
Passing No. 20, retained on No. 30	5	10
Passing No. 30, retained on No. 50	40	80
Passing No. 30, retained on No. 80	15	40
Pan	0	5

The glass spheres have a refractive index between 1.50 and 1.52 and 80 percent minimum overall rounds per screen.

The glass spheres mixed into the paint during the manufacturing process have the following gradations:

U.S. Standard Sieve Size	Percentage Passing
No. 40	100
No. 60	80-100
No. 100	30-50
No. 200	0-5

These glass spheres also have a refractive index between 1.50 and 1.52, and there are 80 percent minimum overall rounds and 70 percent minimum rounds per screen.

The sizes of these glass spheres are shown in Figure 1. This graph shows that glass spheres are available from 3 to 28 mils, with an average of 14 mils for the drop-on glass spheres and 6 mils for the premixed size. Because a glass sphere should be embedded to a minimum of 50 percent of its diameter, a 14-mil glass sphere is the ideal size for a 7-mil dry paint film. However, because of the inconsistent thickness of an applied paint line, larger and smaller glass spheres are available to maximize the retroreflectivity of a thicker or thinner paint line.

Because smaller glass spheres are premixed into the paint, the specific gravity of the paint will increase slightly, and the solids content will also increase. As this happens, the cost per mil of the paint decreases.

Both the standard plain paint with drop-on glass spheres and the premixed dual paint are applied the same way from some form of paint storage tank through a paint gun; the glass spheres are then applied by a bead dispenser. Both paints are applied at a film thickness of 15 mils wet, 4 in. wide, at 10 to 15 mph.

When a 15-mil wet standard paint line dries to 7.35 mils, the traffic paint is 49 percent solids by volume. The premixed paint in North Carolina is 62 percent solids by volume and dries to 9.3 mils. Therefore, a gallon of standard paint (49 percent solids) and a gallon of premixed paint (62 percent solids) would both cover 321 linear ft, 4 in. wide, and 15 mils wet. The area covered is the same but, as mentioned, the standard line would dry to 7.35 mils and the premixed line would dry to 9.3 mils. The premixed line would be 26.5 percent thicker than the standard line, which influences durability and the life of the line.

The standard 15-mil wet line with 49 percent solids by volume, which then dries to 7.35 mils and has all glass spheres smaller than 45 mesh (13.9 mils), can be embedded a minimum of 50 percent in the line, as shown in Figure 2a. If this line is worn to 40 percent of its original 7.35 mils, then only 2.94 mils of paint will remain and all the glass spheres will have been lost, thereby leaving a nonreflective paint stripe, as shown in Figure 2b.

A premixed paint applied at 15 mils wet and 62 percent solids will dry to 9.3 mils, as shown in Figure 2c. When this line is worn to 40 percent of the original 9.3 mils, it will still be 3.72 mils thick and contain glass spheres smaller than 80 mesh (7 mils), as shown in Figure 2d. The data in Figure 1 indicate that drop-on beads are generally larger than 6 mils; this explains why no glass spheres are remaining in Figure 2b. A comparison of the data in Figures 1 and 2d indicates that 5 percent of the drop-on spheres and 79 percent of the premixed spheres are still intact. Therefore, 1.5 billion glass spheres would remain in every 321 linear ft of a 4-in. line. Data for 60, 40, and 20 percent remaining paint on the road surface are given in Table 1. The 9.3-mil premixed line is capable of holding all spheres up to 35 mesh (19.7 mils) at 50 percent embedment. As stated previously, because no application of traffic paint can be controlled to a close tolerance, an extra percentage of larger glass spheres are included for those times when the dry film thickness would reach 14 mils. The thicker the paint film, the larger the glass spheres that should be used as a drop on.

Figure 1. Size of drop-on and premixed glass spheres in relation to paint film thickness.

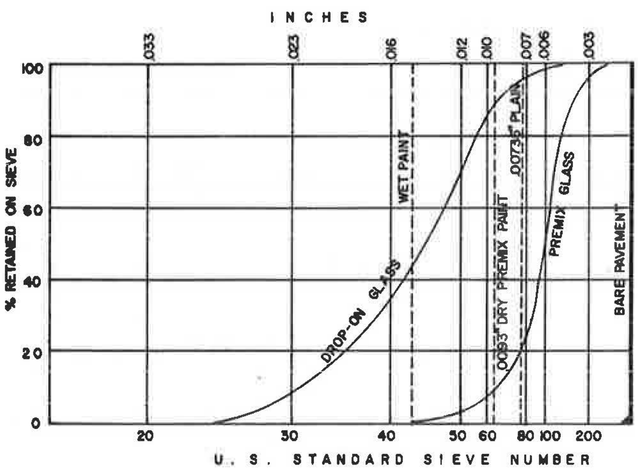


Figure 2. Glass spheres embedded in various dry paint films when new and after wear to 40 percent of original dry film thickness.

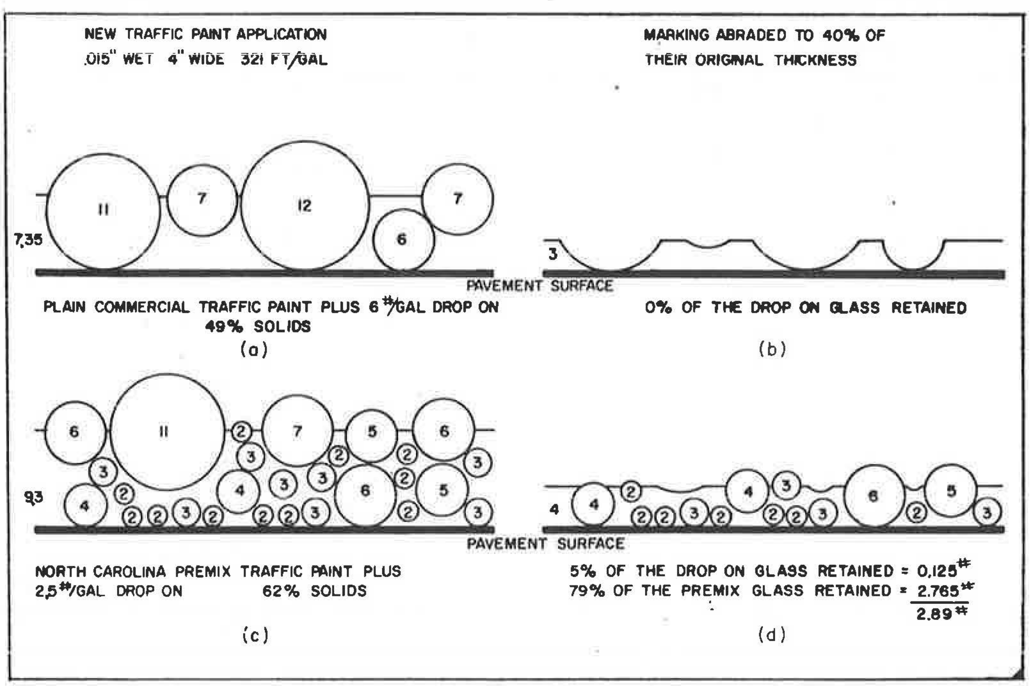


Table 1. Data for remaining paint on road surface.

Item	Plain Traffic Paint Plus 6 lb of Glass Beads per Gallon Dropped on (avg nationwide)	Paint with Premixed Glass Plus 2.5 lb of Glass Beads Dropped on (North Carolina standard)
Percentage of solids by volume	49	62
Dry film thickness, T (in.)	0.00735	0.0093
Maximum size of glass retained [T + 0.5 (in.)]	0.0147	0.0186
Remaining paint on road surface		
60 percent		
Thickness (in.)	0.0044	0.0056
Maximum glass size (in.)	0.0088	0.0112
Glass retained (%)	8 DO	30 DO 97 PM
Weight (lb)	0.47	0.75 DO 3.395 PM
Total		4.145
40 percent		
Thickness (in.)	0.0029	0.0037
Maximum glass size (in.)	0.0058	0.0074
Glass retained (%)	0 DO	5 DO 79 PM
Weight (lb)	0	0.12 DO 2.765 PM
Total		2.890
20 percent		
Thickness (in.)	0.0015	0.0019
Maximum glass size (in.)	0.003	0.0038
Glass retained (%)	0	0 DO 7 PM
Weight	0	0 DO 0.245 PM
Total		0.245

Notes: DO = dropped-on glass beads and PM = premixed glass beads.
It is assumed that all lines are applied at 0.015 wet film thickness, at a 4-in width, for a 321-ft line.

TESTS AND RESULTS

The premixed dual system increases durability, which therefore increases reflectivity. A test of a standard line with drop-on glass spheres was conducted in Alabama, and the standard line was compared with a premixed dual system (5). It was reported that the greater quantity of drop-on beads on the standard line had better initial nighttime visibility, but it had lower durability over a long period of time when compared with the premixed dual system. Also, it was reported that the premixed dual system gave better distribution of the beads in the paint film and had better durability than the standard drop-on line (6). It has also been reported that the main advantage of a premixed line is its greater durability (7).

In order to determine the validity of the North Caroline experience, a series of test lines that used a premixed system were applied next to a standard line in approximately 0.5-mile sections. The standard line consisted of a 15-mil wet commercially available plain paint on which were dropped 6 lb of North Carolina specification drop-on spheres per gallon of paint. The lines were applied on a rural two-lane road to determine the reflectivity and durability under actual driving conditions. These lines were placed so that they were exposed to snow, sand, salt, and studded tires as early as November. A visual evaluation of these lines was made each month by an experienced team of evaluators, which included township policemen and other officials. Photographs and motion pictures were also taken each month. These were used only for illustration, and not for evaluation purposes.

One test area was used to determine if drop-on beads were necessary on premixed lines. A premixed line that did not have drop-on beads applied was placed next to a standard line. The evaluation revealed that, initially, the premixed line did not have satisfactory night visibility when compared with the standard line. Nevertheless, after 8 months of testing, which included 4 winter months, the premixed line was more visible because the standard line was badly worn, as shown in Figure 3. (The premixed line is on the left and the standard line is on the right.)

Another test area was used to determine if the addition of 2.5 lb of glass beads per gallon dropped on the premixed paint increased the night visibility when compared with a standard line. The additional drop-on beads gave greater nighttime visibility than the premixed line without drop-on beads, but it was not as bright as the standard line with 6 lb of glass beads per gallon. The 2.5 lb of beads per gallon droped on the premixed line increased the durability of the premixed line, and after 8 months it was more visible than the standard line, which was more worn. This is shown in Figure 4.

Because North Carolina formulates its own premixed paint, it was necessary to determine whether the advantages with the North Carolina paint were because of the better formulation of quality paint or because of the inherent advantages of the premixed system. A commercially available premixed paint was applied with 2.5 lb of drop-on beads per gallon and compared with a standard line. The test indicated that a commercial premixed paint also had advantages similar to the North Carolina paint, as shown in Figure 5 (the standard line is on the left and the commercial premixed paint is on the right).

North Carolina has not used white centerlines on two-lane, two-way highways since 1970. Figure 6 is a photograph that was taken in July 1982; it shows a white centerline still remaining after 12 years of use (left line of the figure). The other line is yellow paint. Figure 7 shows another road where the white centerline is still intact between the two yellow lines after 12 years in service. Figure 8 shows still another road, and again the white line is still visible. These photographs were taken in 1982, 12 years after the white stripe was applied.

North Carolina has been using the premixed dual system for more than 30 years. Numerous paint tests have proved that the premixed system has a longer life because of the small premixed glass spheres. These small glass spheres provide wear resistance and durability while also providing retroreflectivity down to the last speck of paint on the road surface. The longer life (reflectivity and durability) reduces the repainting cycle needed for an effective line. Thus safety is enhanced because (a) the motorist has continuous day and night visibility and (b) the time necessary for personnel and equipment to be exposed to the hazards of striping highways is reduced. The use of a premixed dual system in North Carolina saves the state money in materials and application costs and also saves the motoring public money by having safer highways.

Several years ago North Carolina found that in painting some 46,000 paved miles of highways and rural streets (12,000 miles of the primary and 34,000 miles of the secondary system), approximately 100 percent of the centerlines and edge lines on the primary system and 73 percent of the centerlines and 21 percent of the edge lines on the secondary system were painted each year. This painting program has been reduced by about 15 percent without an appreciable reduction in the quality of the painted line. For many years North Carolina found that repainting the same line during the same paint season because

Figure 3. Premixed line with no drop-on beads (left) and standard line (right): initially in June (top) and after 8 months in February (bottom).

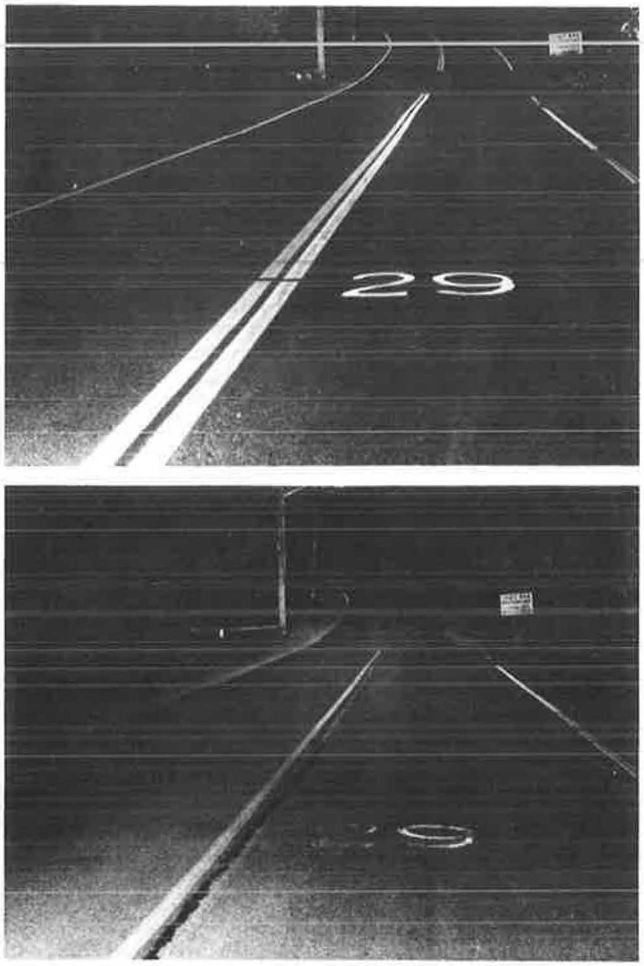
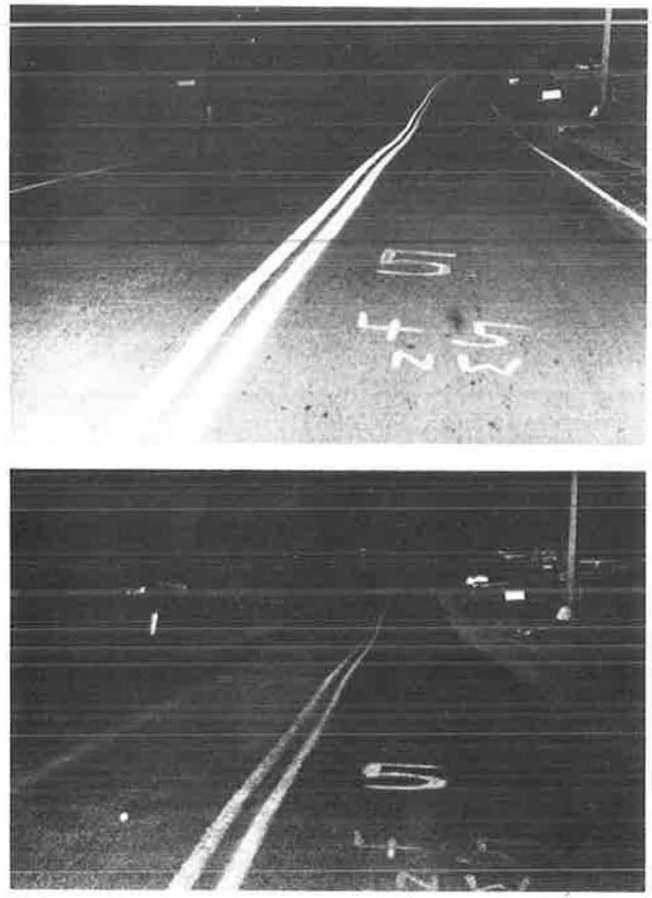


Figure 4. Condition of standard line (left) and premixed dual line with 2.5 lb of glass beads per gallon dropped on (right): initially in May (top) and after 8 months in January (bottom).



of heavy traffic volumes amounted to less than 1 percent during the same season.

By using North Carolina paint costs for plain paint plus 6 lb of drop-on glass spheres per gallon versus premixed paint plus 2.5 lb of drop-on glass spheres per gallon, the premixed system saved a considerable amount of money. Because North Carolina used 840,000 gal of paint during fiscal year 1982, the state saved $193,000 (see Figure 9). Studies in North Carolina indicated that the cost of a traffic painting operation is approximately 21 percent labor, 12 percent equipment, and 67 percent materials. Because of the longer durability of the premixed dual system, there is less need to repaint; therefore, there are lower costs for materials, labor, and equipment.

EQUIPMENT

With all the advantages that can be obtained with a premixed dual system, there are agencies that raise legitimate questions about the system. Based on more than 30 years of experience, the following comments are appropriate.

Generally, a premixed traffic paint must be manufactured by using better raw materials and must be formulated to closer tolerances. This also requires an understanding of the settling that can occur and methods to counteract this behavior. All paints, regardless of their intended use, will settle to some extent with time. Paints that contain a higher percentage of solids will settle more than those paints that contain a lower percentage of solids at the same viscosity. Traffic paints with premixed glass spheres have a tendency to settle a little more; hence there is the need for an antisettling agent to be formulated into the paint.

Any good painting operation, regardless of type, will have a means to agitate or tumble the paint before its use. Traffic painting is no different, and the paint should not be stored for any length of time without agitation. The paint should be used on a first-in, first-out basis and should be thoroughly mixed before and during application.

The equipment required to apply any traffic paint is specialized and expensive. It has to be designed and constructed for the particular operation by using the best materials and workmanship. Once the equipment is placed in operation, it must be well maintained. A major concern of nonusers of premixed paint is the wear on paint gun parts and pumps, and the downtime that would result from this wear. This wear and downtime have amounted to approximately 0.3 percent of the marking program costs in North Carolina. A majority of this expenditure would be required even with a plain paint system, considering that approximately 1 million gal of paint per year is applied on the highways through 28 machines (14 centerline and 14 edge line machines).

The experience in North Carolina has diminished many of the problems with equipment by specifying the following guidelines:

1. The equipment uses a traffic paint with glass beads mixed in the paint;

2. The paint storage tank should be fitted with a removable top, and the bottom should slope to allow drainage for each compartment;

3. A minimum of one hydraulic-driven material agitator that has speed control and a valve shutoff should be used; the props should be as large as practical and of sufficient size to keep the glass and paint pigments in suspension;

Figure 5. Condition of standard line (left) and commercial premixed paint with 2.5 lb of glass beads per gallon dropped on (right): initially in May (top) and after 9 months in February (bottom).

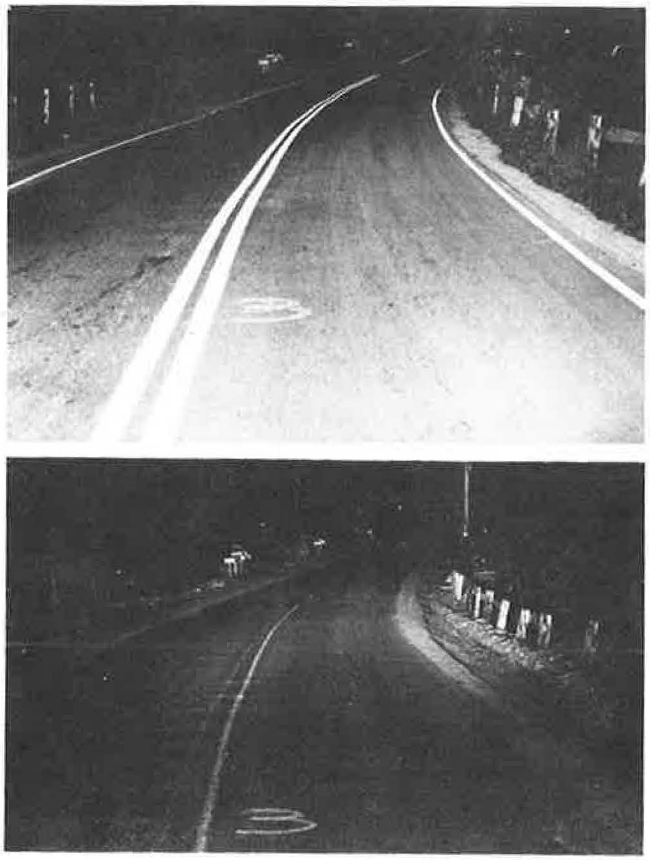
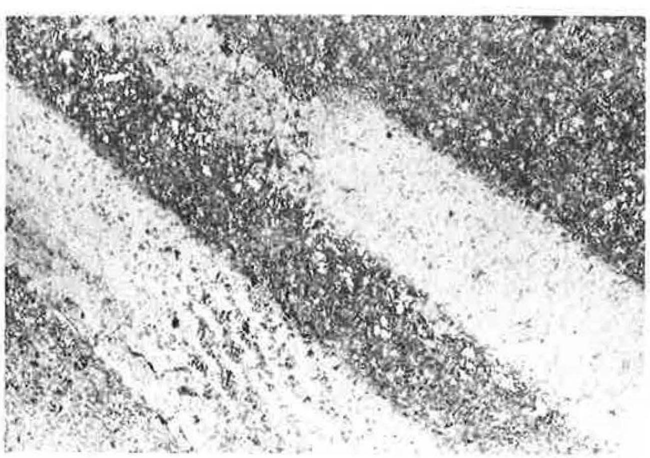


Figure 6. White premixed paint (centerline, upper left) after 12 years of service; rest of line painted over in yellow.



4. Air-operated diaphragm pumps should be equipped with Teflon ball valves and diaphragms; wetted parts should be made of stainless steel; and all material conductors, other than the pipes, should be nonmetallic and flexible and have a solvent-resistant nylon or Teflon base;

Figure 7. White premixed paint (centerline) after 12 years of service, shown between yellow lines.

Figure 8. White premixed paint (centerline) still visible after 12 years of service.

Figure 9. Cost savings.

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PLAIN PAINT + 6#/GAL D.O.                               P.M. + 2.5#/GAL D.O.

WHITE -                  $ 4.08                         WHITE -                  $ 4.69
GLASS 6# X $.23 =        $ 1.38                         GLASS 2.5# X $.23 =      $  .575
                         $ 5.46   DIFF. = $ .185                                 $ 5.275
                                     X  422,395 GALLONS
                                     $   78,143

YELLOW                   $ 4.26                         YELLOW                   $ 4.79
GLASS 6# X $.23 =        $ 1.38                         GLASS 2.5# X $.23 =        .575
                         $ 5.64   DIFF. = $ .275                                 $ 5,365
                                     X  417,525 GALLONS
                                     $  114,819

                                                              $  78,143
                                                              $ 114,819
ASSUME ANNUAL PAINTING                                 TOTAL  $ 192,962  MATERIAL

ADDITIONAL SAVINGS IN LABOR AND EQUIPMENT RENTAL DUE TO NEED TO
PAINT LESS OFTEN. ALSO LESS EXPOSURE OF PUBLIC TO PAINT EQUIPMENT
ON HIGHWAY AND CONTINUOUS REFLECTORIZATION "REDUCES" ACCIDENTS.
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5. The paint conductors, which are rigid paint lines and fittings, should be at least 2-in. standard weight pipe, and all valves in these lines should be of the full-throated ball-valve type with Teflon seats; and the nonrigid paint conductors should be flexible and made of a solvent-resistant material of at least 0.5-in. inside diameter;

6. The truck-mounted guns should be Binks 21 with brass bodies capable of processing premixed material in quantities that will yield a line of 15-mil wet film thickness and a sharp, clean 4-in.-wide line; and

7. Hand spray guns should be a Binks Model 69 equipped with a number 59A fluid nozzle, a number 242 air nozzle, and a number 559 needle.

Another concern of the nonusers of premixed paint is the attitude of the maintenance people. If this group is not fully trained and convinced that their effort is necessary, the best results will not be obtained. Placing a standard line alongside a premixed line in each district would illustrate the results of their care and concern.

Figure 10. Premixed line with 5 lb of glass beads per gallon dropped on (left) and standard line with 6 lb of glass beads per gallon dropped on (right).

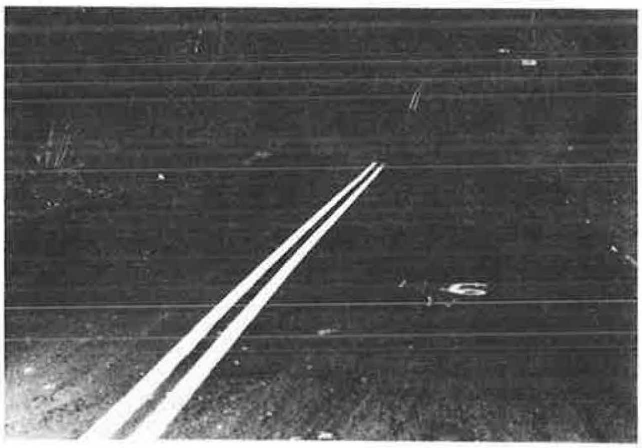
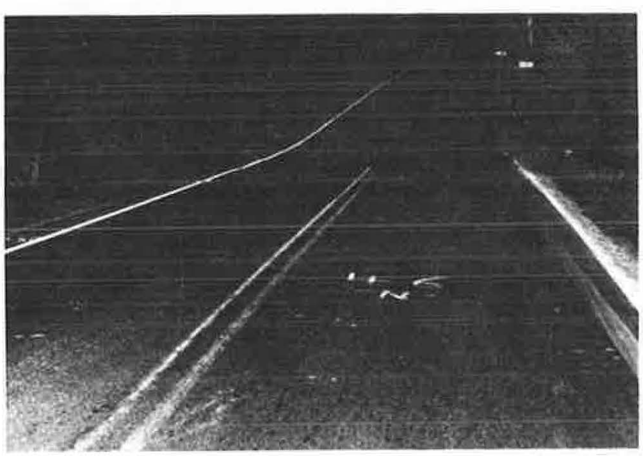


Figure 11. Same as Figure 10, but after being in service.



Because a standard line has 6 lb of glass spheres per gallon on the surface of the line compared with 2.5 lb on the premixed line, the premixed line may not be as visible at night initially. If this is objectionable, it can easily be corrected by applying more drop-on glass spheres. This is shown in Figure 10, where a premixed paint line with 5 lb of glass spheres per gallon dropped on is shown on the left and a standard line with 6 lb of glass spheres per gallon dropped on is shown on the right. Nevertheless, as the line wears, the premixed line retains more visibility compared with the drop-on line, and it can be seen for a longer period because it is more durable, as shown in Figure 11.

CONCLUSIONS

After more than 30 years of experience in North Carolina, the premixed dual system has proved to be a durable reflective system that saves the state money. North Carolina also uses a 3-min premixed dual system that has the same characteristics as the conventional drying premixed paint.

The North Carolina premixed dual system has the following advantages over other systems:

1. Increased service life,
2. Extended night visibility,
3. Rapid drying time,
4. Satisfactory and long-time visibility during the day, and
5. Lower cost.

The cost per foot of applied line per day of useful life should be the criterion for selecting the traffic paints for all agencies. All costs such as material, labor, and equipment must be considered as well as the wear characteristics. The life in terms of longer reflectivity and greater durability is the key to maximizing the traffic markings on streets and highways.

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Abridgment

Collection of Work-Zone Accident Data

JERRY L. GRAHAM AND JAMES MIGLETZ

The objective of this research was to recommend, based on analyses of current practices and the trial implementation of a promising new system, an effective yet simple information procedure that relates accident factors to traffic control deficiencies in highway work zones. The procedure, called the work-zone accident data process (WZADP), is appropriate for two major applications: (a) it provides information that can be used to determine if a correction or change is needed in the traffic control at the work site where the accident occurred, and possibly the type of change indicated; and (b) it provides information that can be combined with that from other sites to form a data base from which accident trends and the relations between accidents and the various traffic control devices and strategies can be determined. In the first phase of this research a user's manual was developed that described the WZADP. This manual was used in trial implementations in Iowa and North Carolina. The results of the trials indicated that the WZADP is a useful procedure for collecting work-zone accident data and that, in general, the safety record of the work zones monitored was extremely satisfactory. Problems were noted in some discrepancies between the number of accidents reported under the WZADP and other state accident records systems, and in some instances the police did not report all work-zone accidents to project managers.

Traffic safety in highway work zones has drawn increasing emphasis because of an alarming number of lawsuits involving construction-zone fatalities and the growing need for maintenance of highway facilities under traffic. The most direct indicators of the level of traffic safety in work zones are work-zone accidents.

In 1976 Midwest Research Institute (MRI) undertook a project that examined the problems of work-zone traffic control from both safety and operational viewpoints. The final report of this project was entitled, "Accident and Speed Studies in Construction Zones" (1). During this project an accident data base was formed by using data from 79 construction projects in seven states. From the analysis of this data base, several recommendations were made concerning the safety of work-zone traffic controls. For example, it was found that the accident rate is much higher on six- or eight-lane freeways when two lanes are closed rather than a single lane. Also, during the analysis of hard copy accident reports for three projects, it was observed that there was a predominant accident type throughout the project duration that remedial actions could have alleviated.

On October 13, 1978, FHWA issued Federal-Aid Highway Program Manual (FHPM) 6-4-2-12 (2), a regulation pertaining to all Federal-Aid construction projects. This regulation contains many procedures that assure that adequate consideration is given to the safety of motorists, pedestrians, and work-force personnel.

Two mandatory procedures described in the regulation are as follows.

1. The highway agency shall designate a qualified person who will have primary responsibility and sufficient authority to assure that the traffic control plan and other safety aspects of the contract are effectively administered:
    a. On small projects this person may be the resident or project engineer;
    b. On large projects this person should be a designated person who is trained in traffic control measures; the individual would be assigned to this task on a full-time basis and would not be responsible for other duties; and
    c. Such persons may be responsible for one or more projects, depending on project magnitude and proximity to other projects.
2. Work-zone accidents shall be evaluated on both a short-term and long-term basis:
    a. On a short-term basis the responsible person makes arrangements to obtain accident information, including accident locations, as quickly as possible so that current operational problems can be evaluated and appropriate changes can be made in the traffic control plan; and
    b. On a long-term basis a sampling of accidents that occur in work zones is identified and analyzed; this information should be used to correct deficiencies in traffic control standards and to improve the content of future traffic control plans.

OVERVIEW OF RESEARCH

The results of an FHWA research project entitled, "The Collection of Work-Zone Accident Data," are described in this paper. The project was based on the idea that further gains in work-zone traffic safety could only be obtained if states have reliable work-zone accident data bases. These data bases must contain enough information about the work-zone traffic controls to establish the relationship of these controls to accidents that occur in the work zone.

The project work plan was comprised of two phases. The first phase involved a survey of the current practices in nine states, the development of three alternative reporting procedures, and the preparation of a user's manual describing one or more of the reporting procedures. The second phase was a trial implementation of the accepted reporting procedure in two states. The focus of this paper is on the effectiveness of the work-zone accident data process (WZADP) during the trial implementation in Iowa and North Carolina.

[The user's manual developed during this project has been published as Report FHWA-IP-82-15, "Work-Zone Accident Data Process-User's Manual," and is available from the U.S. Government Printing Office. The project final report is Report FHWA/RD-82/501, "The Collection of Work-Zone Accident Data," and is available from the National Technical Information Service, Springfield, Virginia.]

TRIAL IMPLEMENTATION RESULTS

The ultimate goal of the WZADP is to reduce accidents in highway construction and maintenance projects by identifying potential hazards and problems and correcting them. It may be too early to determine all of the safety benefits to be derived from this research, although the safety records of the projects involved in the trial implementation are known. The results of the trial implementation are discussed in this section.

Safety

Forty projects were studied in Iowa. A summary of

Table 1. Iowa work-zone accident summary.

Project Type	No. of Projects Studied	No. of Accidents Before Construction	Before Accident Rate	No. of Accidents During Construction	During Accident Rate
Interstate	10	42	37[a]	11	95[a]
Primary route	26	42	70[a]	7	26[a]
Intersection	4	91	5.79[b]	60	3.89[b]
Total	40	175		78	

[a]Accidents per 100 million vehicle miles.
[b]Accidents per million entering vehicles.

Table 2. North Carolina accident data summary.

Project	No. of Accidents Before Construction			Before Accident Rate	No. of Accidents During Construction			During Accident Rate
	Fatal	Injury	PDO		Fatal	Injury	PDO	
1	3	146	329	327.5[a]	0	13	38	202.4[a]
2	3	28	63	55.7[a]	0	6	15	65.3[a]
3	0	7	24	2.44[b]	0	3	7	2.40[b]

[a]Accidents per 100 million vehicle miles.
[b]Accidents per million entering vehicles.

Table 3. Iowa accident reporting discrepancies.

Project	Accidents Reported Under WZADP	Accidents Reported by ALAS
1	4	8
2	16	42
3	16	34
4	24	12

Note: ALAS = accident location analysis system.

the accident rates before and during construction of these projects is given in Table 1. Intersections and primary route projects both had substantial decreases in accident rates. Interstate accident rates increased during construction, but the severity of the accidents was low [all property-damage-only (PDO) accidents]. Analysis of the data in Iowa revealed few changes in accident type occurring during construction. Most of the work-zone accidents occurred in the work area.

A summary of the accident experience on three North Carolina projects is given in Table 2. The first project had a significant decrease in the accident rate during construction, and the second and third projects did not change significantly. There were no fatal accidents on the projects during construction.

Overall, the review of the work-zone accident data in both states revealed that traffic controls performed satisfactorily.

Accident Data Discrepancies

In both states different accident reporting systems yielded different numbers of work-zone accidents. A comparison of the number of accidents reported by project personnel versus the number of accidents found in the computerized accident records system for four projects in Iowa is given in Table 3. For three of the projects, the number of accidents reported through the WZADP was much lower than the number found in the computerized system, but the fourth project had more accidents reported by the WZADP.

In North Carolina the number of accidents reported by the WZADP was much lower on all three projects. In general, project managers in North Carolina were aware of accidents that occurred near the work area; they were not aware of other accidents that occurred within the project boundaries that were not near the immediate work area.

Part of the reason for discrepancies could be that police officers did not notify project personnel of accidents within project boundaries that were not work related or that did not occur near any work activity.

Hazards Identification

The formalized procedures for identifying potential hazards presented in the user's manual required project personnel to identify problems in a logical manner. It is believed that the participants in the trial implementation will retain this experience and, by using the user's manual (or other similar publication) as a reference, will use the experience to identify and reduce hazards in future construction and maintenance projects.

In both Iowa and North Carolina, hazard identification led to changes in work-zone traffic control procedures. Procedures for operating pilot cars on rural routes in Iowa were changed after a rear-end accident occurred and other traffic problems were identified. At a high-volume lane closure in North Carolina, the location of an arrow board in advance of the taper was changed as a result of rear-end accidents and numerous skid marks.

Incidents

North Carolina formally did not plan to collect incident data. Nevertheless, examination of accidents and skid marks (a type of incident) was needed to solve a traffic control problem. In Iowa the collection of incident data at the project level proved useful mainly in rural areas where the number of accidents was low. In urban areas project managers at first tried to document every minor incident and found that too much time was required.

Because of those initial problems, no data on incidents were collected in urban areas, even though project managers knew about some unreported accidents.

Amount of Additional Work for Project Managers

There is no doubt that the project managers were required to do a lot of extra work during the trial implementation. Accident and incident reporting and investigation took up to 2 hr per accident or incident. Time was also spent going to the local police departments to obtain copies of the accident investigation reports. Time was also spent observing traffic driving through work zones. Nevertheless, in many states this observation is part of standard departmental procedures. The procedure of identifying potential hazards by observing traffic was probably improved by the training the project managers received.

Transmittal of Information

In Iowa information transmitted through reports submitted by the resident engineers to the district office worked satisfactorily. In North Carolina the project managers were not always sure of how the information was transmitted or who received the information. Accident reports were to go from the project manager through the district traffic engineer to the state traffic engineer. Not all of the requirements of reporting and summarizing accidents as specified in the training were fulfilled in North Carolina. Note that these same requirements are standard procedures in North Carolina, which means that some persons were not complying with departmental requirements.

Training

The people who received the training believed it was beneficial to understanding their responsibilities. Some also believed that the information that did not directly pertain to them (such as project and statewide accident summaries) was of little benefit.

Some type of training is required for all people who participate in the WZADP. The participants believed that training could be taught by persons within the highway agency. The training should be informal and could be done at the district level. Headquarter officials, who would probably present the training, would need an indoctrination before presenting the WZADP at the districts.

Police Cooperation

In both states police agencies agreed to cooperate in the trial implementation. Nevertheless, project managers were not always notified of accidents that occurred within project limits. Part of this may be due to the training the police received and their definition of a work-zone accident. An accident was usually noted as a work-zone accident by a police investigator if there was a definite relation to the work activity. It may be that project managers will have to put forth extra effort in order to learn about all work-zone accidents. It appears that the police agencies will continue reporting work-zone accidents as they have been doing.

EFFECTIVENESS OF WZADP

The ultimate goal of the WZADP is to eliminate work-zone accidents and increase the efficiency of traffic control procedures. In both states the trial construction projects were no more hazardous than the roadways were before construction began. This low accident record may have been because of the WZADP or because Iowa and North Carolina may be especially aware of the need for satisfactory work-zone traffic controls. Nevertheless, the real proof of success will be in the accident records of future construction and maintenance projects. If the WZADP is worthwhile, accident records will substantiate this fact.

Project managers have stated that the formalized process of identifying and correcting potential hazards has been a benefit to them. These people now know how to identify and correct potential hazards, and they will continue to use the procedures specified in the user's manual.

The headquarter personnel of both states that managed the trial implementation were supportive of the project and believed that the procedures are beneficial in increasing safety in highway work zones. Each state highway agency has its ideas on managing the WZADP and will continue this program in a way that meets the needs and requirements of their individual states.

CONCLUSIONS

1. The WZADP has proved to be a usable procedure for collecting work-zone accident data and analyzing these data to make needed corrective changes in work-zone traffic control.
2. Because there is a great deal of variability in the organization, personnel, and facilities of the various state highway departments, there is a need for a great amount of flexibility in the design of the WZADP.
3. Some states may wish to implement the WZADP on special projects or in selected areas only. This approach could also be used to implement the WZADP in stages.
4. Project-level personnel who are expected to record work-zone accident data should be trained by using the WZADP user's manual.
5. Care must be taken in specifying actions required of project personnel. If the work load is excessive, parts of the procedure may be abandoned.
6. Police cooperation should be established at the headquarters level and reconfirmed with investigating officers. Experience in phase 2 of this project revealed that cooperation will normally only extend as far as furnishing reports that are requested. Reliance should not be placed on officers informing someone that an accident occurred. The project manager will normally have to contact police agencies at regular intervals to obtain accident reports.
7. Discrepancies in the number of accidents reported under the WZADP and other state accident records systems were discovered in both Iowa and North Carolina. These types of cross-checks should be made to ensure that all regularly reported accidents within work zones are coming to the attention of project managers.

ACKNOWLEDGMENT

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We gratefully acknowledge the assistance of state officials in Alabama, California, Connecticut, Nebraska, New York, Ohio, Utah, Virginia, and Wisconsin during the first phase of the project, and of state officials in Iowa and North Carolina during the second phase of the project.

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Notice: The opinions expressed in this paper are those of the authors and not necessarily those of FHWA or any of the cooperating state agencies.

Field Evaluation of Snowplowable Pavement Markers

JERRY G. PIGMAN AND KENNETH R. AGENT

The objective of this study was to evaluate available snowplowable markers under similar traffic and snowplow operations. Five different markers were tested: Stimsonite 96, Dura-Brite, recessed, Kingray, and Prismo roadstud. The evaluation revealed that the Stimsonite 96, Dura-Brite, and recessed markers were acceptable snowplowable markers, because all three had adequate reflectivity during both dry and wet nighttime conditions. This reflectivity was maintained over the test period, and the markers proved to be durable when subjected to snowplow operations. Nevertheless, considering all available input, the recessed marker is recommended as the most functional and cost-effective marker.

Raised pavement markers have proved to be an effective delineation treatment during wet nighttime and poor visibility conditions, especially in states outside of the snowbelt. Nevertheless, the problems that result from snowplow operations are particularly severe, and marker applications are limited. Even in a border state such as Kentucky, where more than 1 million raised pavement markers have been installed, only one winter of heavy snow and resultant snowplow operations can destroy a significant part of the installations.

In an attempt to provide wet nighttime delineation by using the concept of raised pavement markers, considerable effort has been devoted to developing snowplowable pavement markers. The most widely used and most successful approach to the development of a snowplowable marker has been to retain the reflective unit of a raised pavement marker and attempt to protect it from snowplows. Usually the reflective unit is encased or surrounded by a material that is resistant to snowplow blades. Consistently mixed results, particularly with regard to the cost-effectiveness of the markers, have been the rule in almost all experimental and large-scale installation projects.

Several types of snowplowable markers have been field tested in the past few years. These tests have been conducted independently under different field conditions. The objective of this study was to evaluate all available snowplowable markers under similar traffic and snowplow operations.

BACKGROUND

A recent survey (1) of the use of snowplowable markers indicated that the majority of existing markers were the Stimsonite marker--either the Stimsonite 96 model or the older Stimsonite 99 model (see Figure 1). This marker consists of an iron casting with an attached prismatic retroreflector. Both ends of the castings are shaped to deflect a snowplow blade. This marker has been evaluated (2-4), but it had not been compared directly with other markers.

The survey indicated that several states experimented with a recessed marker (1). The installation in this study involved placing a regular or low-profile raised marker into a groove cut into the pavement so that the top of the marker was flush with the pavement surface. A recessed marker, which used a regular raised marker in the groove, was included in this study. Some installations have used a groove with a cross section that had several peaks and valleys (5,6). However, this study used a full-width groove similar to installations in Tennessee and South Carolina. The Stimsonite 911 marker (Figure 2) was installed in the groove.

In an effort to include all other available snowplowable markers in the test, various manufacturers were contacted. As a result, two additional markers were included in the original installation, and a small number of another marker were installed shortly thereafter. The new markers were the Dura-Brite (Figure 3), Kingray (Figure 4), and Prismo roadstud (Figure 5) markers. The Dura-Brite marker includes a steel frame set in precast concrete. The replaceable reflector is mounted between the two steel runners that protrude above the pavement surface. The runners are shaped so that the marker can be plowed at an angle. The Kingray marker involves placing the reflective lens in an insert that is depressed in an outer sleeve when struck by a tire or a snowplow blade. The Prismo roadstud is a die cast aluminum marker that provides an anchor stem for additional durability.

A few other potential snowplowable markers were investigated. However, the development or marketing of these markers had either stopped or was progressing so slowly that they were not available for testing.

The lane-delineation survey also obtained information about installation costs (1). The average cost of numerous installations of Stimsonite markers was approximately $16 per marker, but a more accurate current cost would be close to $20 per marker when installed in large quantities. Cost data were not available for the Dura-Brite marker at the time of the survey, but estimates place the cost of this marker to be similar to the Stimsonite markers. No cost figures are available for large installations of the Kingray marker, but its cost would not be less than that of the Stimsonite or Dura-Brite markers. The most inexpensive snowplowable marker installed to date has been the recessed marker, which has a reported cost per marker in the $8 to $9 range. The cost for a regular, raised pavement marker is approximately $3 per marker.

INSTALLATION

Four of the test marker types were installed in December 1980. The fifth type, the Prismo roadstud, was installed by the manufacturer in January 1981. A contract was awarded for the installation of 150 each of the Stimsonite 96, recessed, Dura-Brite, and Kingray markers. The contract was for $31,371.12, or $52.29 per marker. Installation of such a small number of markers resulted in this extremely high cost. Fifty-two of the Prismo markers were installed at the expense of the manufacturer.

Two test locations were selected. Both locations were four-lane divided highways. One location (US-68 in Fayette County) had a portland cement concrete pavement, whereas the other (US-27 in Jessamine and Garrard Counties) had a bituminous pavement. The following criteria were used when selecting the test locations:

1. The roadway could be plowed with any type of snowplow blade that is used in normal snowplow operations;
2. A minimum annual average daily traffic (AADT) of 15,000 was preferable;
3. Part of one test section should be in a high weave area; and
4. Test sections should not have roadway lighting.

The markers were only to be installed on skip lines.

All snowplow operations were performed with a steel blade. In the past, rubber-tipped blades have been used on roadways that had raised markers. Also, virtually all multilane highways in Kentucky that did not have roadway lighting have had raised markers added. This meant that the snowplowable markers had to replace regular raised markers. For practical reasons, isolated, short sections of mul-

Figure 1. Stimsonite 96 marker.

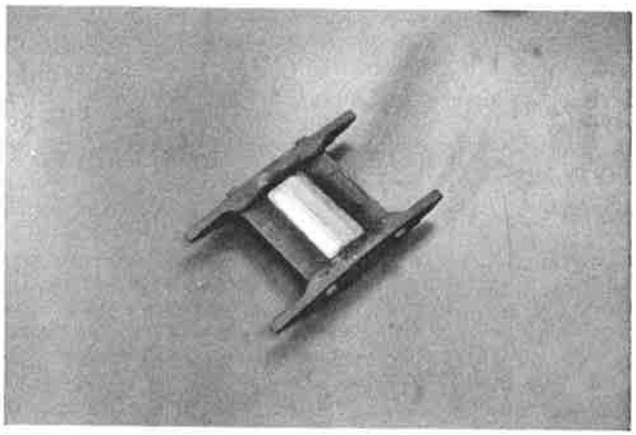


Figure 2. Stimsonite marker used as a recessed marker.

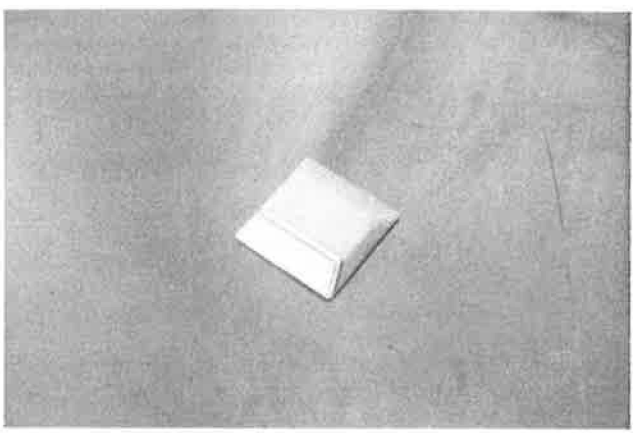


Figure 3. Dura-Brite marker.

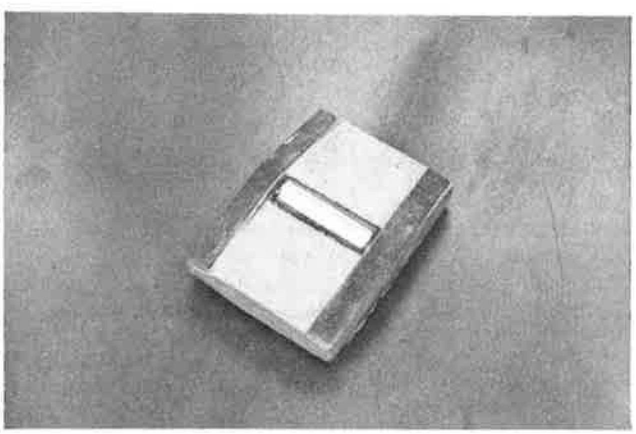


Figure 4. Kingray marker.

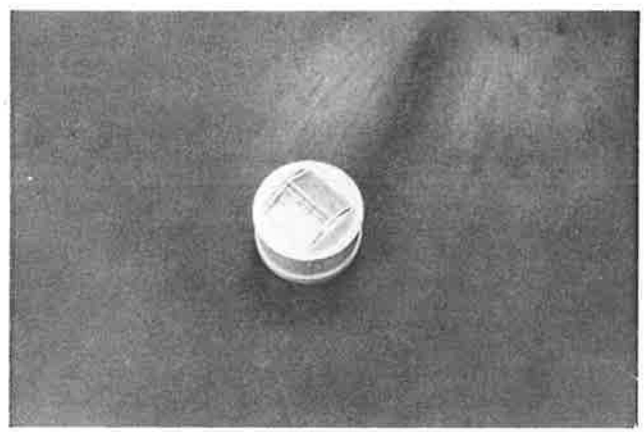
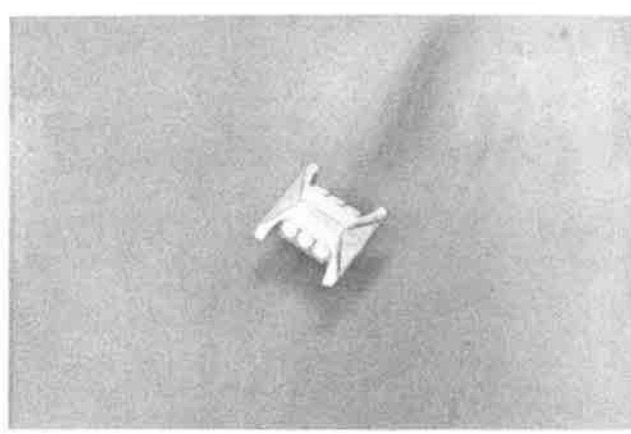


Figure 5. Prismo marker.



tilane highways had to be found for the test installation because maintenance personnel could not be expected to use a different snowplow blade for a short section of a long multilane highway. Arrangements were made with maintenance personnel to assure that the two short sections of highway would be plowed with the normal steel blade.

Both test sections were in areas that did not have roadway lighting. The Fayette County location was adjacent to an interchange and contained several access points, which generated a significant amount of lane changing. The 1980 AADT of the Fayette County location was 16,400, whereas the AADT at the Jessamine-Garrard County location was 7,000. The Jessamine-Garrard County location included a section with a substantial grade. Markers were placed on both the uphill and downhill grade. The old regular markers were removed before the installation of the snowplowable markers.

In general the installation pattern involved alternating the markers so that every fourth or fifth marker was the same. The exception was one direction at the Fayette County location where several of each marker type (22 or 23) were placed together. This was done so that a comparison between the number of markers visible in a line could be made. Also, a regular Stimsonite 911 marker was placed in the pattern in one direction at the Fayette County location. All markers were installed at a 40-ft spacing.

Installation of each of the markers required either a saw cut or a drilled hole in the pavement. The cuts for the Stimsonite 96, recessed, and Dura-Brite markers were made by using diamond-tipped sawblades. The Kingray and Prismo markers required drilling holes in the pavement. The average times for cutting or drilling, installing the marker, and for the adhesive material to dry are given in Table 1. Sawing or drilling time for the Stimsonite 96, recessed, and Prismo markers should be representative of larger installations. However, sawing and drilling time for the Dura-Brite and, in particular, the Kingray markers would be less on larger installations where better procedures could be used.

The time needed to install the markers in the prepared cut would also be less in a large-scale operation. The time needed to install the markers was longest for the Kingray markers and shortest for the recessed markers. The factor that contributed most to the longer time to install the Kingray marker was a requirement that the marker be held in position until the bitumen hardened enough such that the marker would not rotate out of alignment. The longest drying times were for the Stimsonite 96 and recessed markers where epoxy was used. Much shorter drying times were found for the Kingray and Prismo markers, which used a bituminous material, and for the Dura-Brite marker, which used a material called SET-45 (a magnesium phosphate cement).

Table 1. Installation times.

Type of Marker	Time (sec) to Saw or Drill: Concrete	Time (sec) to Saw or Drill: Bitumin	Time to Install Marker (sec)	Time for Adhesive to Dry (min)
Stimsonite 96	12	9	60	60
Recessed	40	25	20	60
Dura-Brite	40	25	90	15
Kingray	720	360	300	10
Prismo	90	90	30	10

Figure 6. Installation at Garrard County, southbound (order of markers is Kingray, Stimsonite 96, Dura-Brite, recessed, and Prismo).

RESULTS

The results consisted of an evaluation of the reflectivity and durability of the markers. The markers were evaluated for a 16-month period after installation. Day and night inspections were conducted quarterly. Additional inspections were made after snowplow operations. There was no significant snowfall requiring snowplows in the first winter, so a snowplow test on wet pavement was made over a portion of the test installation. There were snowplow operations during the second winter, which resulted in the markers being subjected to a total of six to eight snowplow passes. The visual inspections were supplemented with photographs.

Reflectivity

Nighttime observations were made immediately after installation and then on a quarterly basis. Photographs were taken during each inspection.

The first inspection of the four original markers, which was done immediately after installation, indicated that all markers were extremely effective. Observations of the Prismo markers indicated that this marker was also effective. Although the Prismo marker was not as reflective as the others, it still provided adequate delineation and was particularly effective on curved sections.

Results of the periodic nighttime evaluations of reflectivity indicated that most of the marker types maintained satisfactory reflectivity during the test period. The only marker that suffered a substantial loss of reflectivity was the Kingray marker because dirt and water apparently penetrated into the clean air space behind the lens, which resulted in the lens having a foggy appearance. The loss of reflectivity occurred after only a few months. The manufacturer indicated that this problem was overcome by increasing the weld zone of the lens to the backplate and by improving the flow of polypropylene material. However, new markers with this improved feature were not available for testing.

Installing the markers in the alternating pattern allowed comparisons of relative reflectivity. Photographs taken at the southbound Garrard County installation at periodic intervals during the evaluation period give a comparison of all five markers (Figure 6). The Kingray marker had lost its reflectivity visibility, and the Prismo marker was the least reflective marker. The remaining marker types (Stimsonite 96, Dura-Brite, and recessed) demonstrated similar reflectivity.

A photograph of the southbound Fayette County installation gives a comparison of the Stimsonite

96, recessed, and Dura-Brite markers with a regular Stimsonite 911 marker placed on the pavement surface (Figure 7). The comparison indicated that each of these three snowplowable markers had a reflectivity similar to the regular, raised pavement marker.

Observations during wet nighttime conditions were made, and the same general conclusions were found. Particular attention was paid to whether the groove in which the recessed marker was placed would fill with water during wet weather conditions. If this occurred, a loss of reflectivity would result. In all but heavy rains the groove remained relatively dry because of the effect of vehicles passing the marker and the water being vacuumed or blown out. The groove did maintain a level of water for a short time during heavy rains, but this only caused a problem when the geometry of the roadway was such that the marker was on the downhill end of a groove. Overall, it appears that there was no significant problem with the groove becoming filled with water during wet weather conditions.

The visibility of the recessed markers during snow and ice conditions was also observed. After a snowplow operation the groove would be filled with snow and ice. The snow and ice would usually melt in a relatively short period of time, and the resulting water would be swept from the groove by traffic. Some inspections found the groove to be partly filled during these conditions. Approximately the top third of the marker would be cleansed by tires, but the bottom portion would be obscured. This reduced nighttime visibility, but the markers could still be seen. Overall, it was concluded that the recessed marker remained effective during snow and ice conditions. Special attention was directed to the visibility of recessed markers; however, it was noted that light snowfall and ice did not inhibit the performance of the other marker types.

In April 1982, after 16 months in service, the reflective lens of three each of the Stimsonite 96, Dura-Brite, and recessed markers were removed from the field sites for laboratory tests. These reflectors would have initially met the reflectivity requirements in Kentucky for a highly reflectorized marker. The minimum specific reflectivity requirement for a silver-white lens at a 0.2° divergence angle and 0° incidence angle is 2.7 candlepower per footcandle per unit marker. Laboratory tests revealed that the average specific reflectivity for the markers after slightly more than 1 year in service, given in terms of candlepower per footcandle per unit marker, was 2.5 for the recessed reflector, 2.1 for the Dura-Brite reflector, and 1.3 for the Stimsonite 96 reflector. The Dura-Brite and Stimsonite 96 markers use the same reflector. These readings are in agreement with the observed durability of the reflectors in these markers. The lens in the recessed and Dura-Brite markers received little damage, whereas the Stimsonite 96 had some damage. This damage was probably related to the higher profile of the Stimsonite 96 marker. Nighttime observations indicated that all three markers maintained satisfactory reflectivity after 16 months in service. Because of the difficulty of removing the reflective units and their damaged condition after 16 months, neither the Kingray nor the Prismo marker was tested.

Durability

Evaluation of the durability of the markers involved two areas. First, an effort was made to determine the effect of traffic on marker durability, and second, the effect of snowplow operations was evaluated. Most of the markers were not involved in snowplow operations for slightly more than 1 year after installation, which enabled researchers to make an assessment of the effect of traffic on the durability of the markers.

Traffic Wear

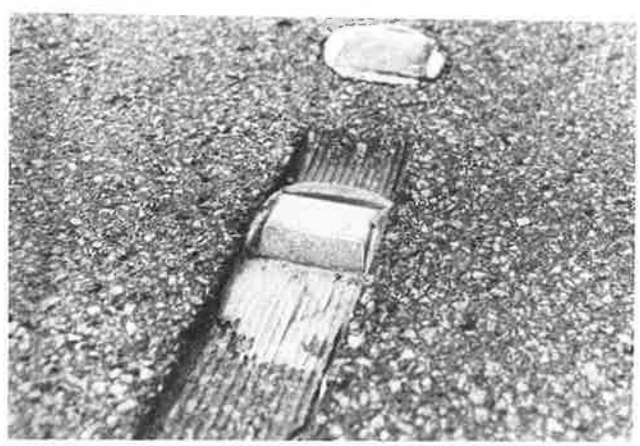
Photographs of the various markers after almost 1 year in service are shown in Figures 8-12. These photographs were taken before the second winter and therefore show the effects of traffic wear only. The summary of marker damage which follows applies to the effect of approximately 1 year of traffic wear with no snowplow damage.

The recessed marker is shown in Figure 8. This marker demonstrated satisfactory durability. Minor damage to the top of the lens was found on seven markers (5 percent). Inspections during the year revealed that the groove remained relatively free of debris, and approximately the top half of the lenses remained clean. The bottom half of the lenses was not satisfactorily cleaned by tires. Also, the abrasive coating on the top half of the lenses was chipped more than the other snowplowable markers.

The Dura-Brite marker is shown in Figure 9. The durability of the Dura-Brite marker to traffic wear was satisfactory. The lenses remained clean, and there was less chipping to the abrasive coating than with the other markers. In some instances the adhesive holding the lens flowed up the covered part

Figure 7. Installation at Fayette County, southbound (order of markers is Kingray, Stimsonite 911, Stimsonite 96, recessed, and Dura-Brite).

Figure 8. Recessed marker after approximately 1 year of service (before snowplow operations).



of the lens. This was caused by the use of butyl tape, which was too thick. The thickness of this tape has since been reduced by the manufacturer. It was also noted that the lens was loose in two markers.

The durability of the Stimsonite 96 marker after being subjected to traffic was also satisfactory (Figure 10). Minor damage to the lens was noted on 13 markers (9 percent). As shown in Figure 10, this damage was minor and did not adversely affect reflectivity. The lens remained clean, but there was minor chipping of the abrasive coating.

Several problems were found with the Kingray marker (Figure 11). The bitumen material holding the marker cracked, and in many instances a large amount of this material was lost, which reduced the bond of the marker to the pavement. A possible reason for the loss of bitumen was failure to heat the hole to a sufficiently high temperature during the installation process. Six markers (4 percent) were missing after almost 1 year in service. The lens also tended to remain dirty because tires would depress and not clean the lens. It was found that rain was necessary to clean the lens. Because the lens did depress on impact, it sustained less abrasive damage to the lens surface than the other markers. About 15 percent of these markers had damage either to the lens or marker. All but two of the markers still recoiled as designed.

Figure 9. Dura-Brite marker after approximately 1 year of service (before snowplow operations).

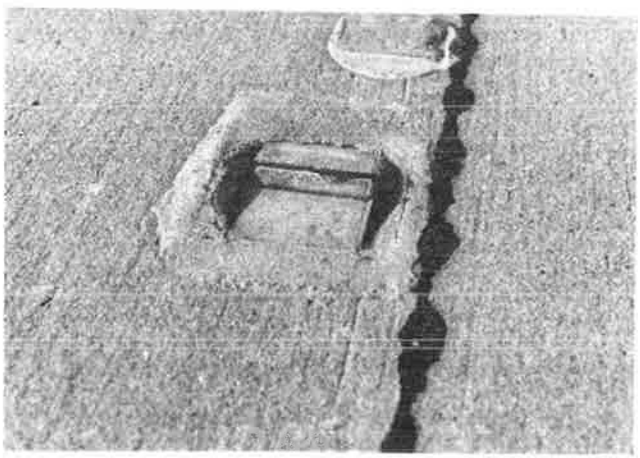
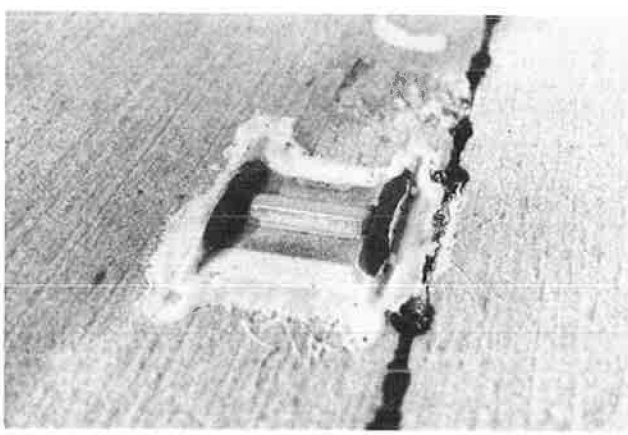


Figure 10. Stimsonite 96 marker after approximately 1 year of service (before snowplow operations).



The Prismo markers at the Fayette County site were removed by snowplows, but observations of the markers at the Garrard County site were made (Figure 12). Five of the markers (17 percent) were missing. The remaining markers were generally in satisfactory condition. Several had minor damage to some of the glass lenses.

Twenty Stimsonite 911 markers were installed at the Fayette County site as a comparison to the snowplowable markers. After almost 1 year of service one of these markers was missing and one had major damage to the lens. There was significant chipping of the abrasive coating on the markers, but they generally remained in satisfactory condition.

Snowplow Damage

During December 1981 and January 1982 between six

Figure 11. Kingray marker after approximately 1 year of service (before snowplow operations).

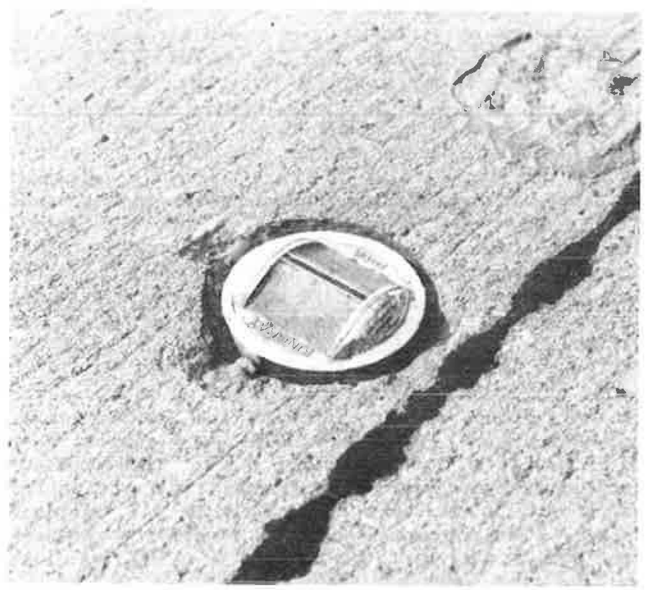
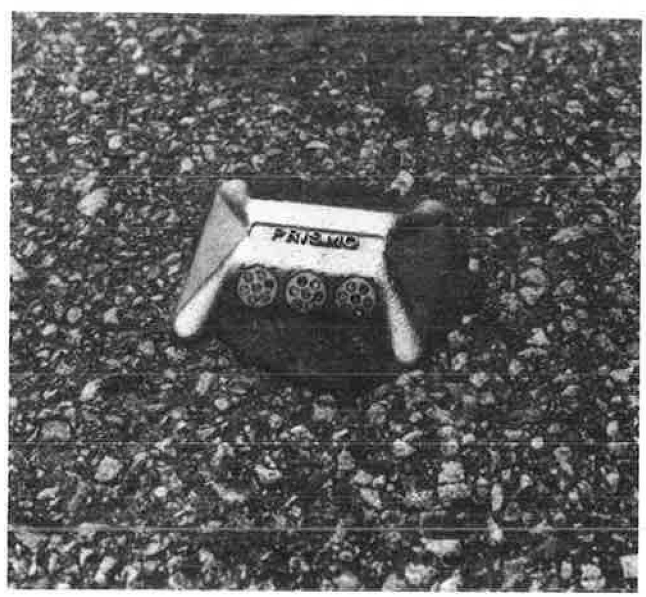


Figure 12. Prismo marker after approximately 1 year of service (before snowplow operations).



and eight snowplow passes were made over the various test sections of markers. A steel blade was used during all operations. The only other snowplow tests were made during January 1981 when two passes were made on the northbound Fayette County location on a wet pavement. In the January 1981 test the Prismo markers were removed and there was damage to three of the Kingray markers (14 percent), whereas the Stimsonite 96, Dura-Brite, and recessed markers proved to be snowplowable, and there was no damage.

A summary of the performance of the markers as a result of the snowplow operations during December 1981 and January 1982 follows. The final inspection was conducted in April 1982 after approximately 16 months in service. The recessed marker was filled with snow after the snowplow operations, but the snow melted and the marker was visible again within a few hours. The recessed marker sustained no additional damage as a result of snowplow operations. Neither the Stimsonite 96 nor Dura-Brite markers sustained any damage to either the lens or the marker housing unit from the snowplow operations. The final inspection indicated that 13 Stimsonite 96 markers and 1 Dura-Brite marker had minor damage to the lens, which was the result of traffic wear. Also, in two of the Dura-Brite markers the lens was missing.

The test indicated that the Prismo marker was not snowplowable. The snowplow sheared the marker off the pavement at the top of the anchor stem. Virtually every Prismo marker involved in the snowplow operation was removed. Also, all of the regular Stimsonite 911 markers that were placed on top of the pavement were severely damaged.

The Kingray markers were also damaged by snowplow operations. Even before the snowplows were used, several of the Kingray markers were either missing or damaged. An inspection after the snowplow operations revealed that 71 Kingray markers (47 percent) were missing, 43 (29 percent) were severely damaged, and 20 (13 percent) were moderately damaged. Only 11 percent were undamaged, and these remaining markers still recoiled as designed.

Another feature of the markers relative to snowplow operations was their interference with snowplow operations. This involved discomfort to the snowplow operator, which resulted from the jolt of hitting the marker, as well as damage to the snowplow blade. The Stimsonite marker, which had the highest profile above the pavement, caused the most interference. The snowplow blade would jump several inches above the pavement after striking a Stimsonite marker. The lower-profile Dura-Brite marker caused less interference. The Kingray and, in particular, the recessed markers caused no interference. The test section was not long enough to show damage to the snowplow blade, but the potential for such damage was demonstrated.

SUMMARY

Installation

All of the markers were installed with relatively few problems. The Stimsonite 96 marker required the shortest saw or drill time. The lengthy drilling time for the Kingray marker would be shortened substantially with better equipment. A more efficient procedure for installing the Dura-Brite markers has been developed by the manufacturer, but it was not used because of the small installation. The time needed to install the markers was highest for the Kingray markers and shortest for the recessed markers. The Stimsonite 96 and recessed markers required longer adhesive drying time because epoxy was used.

Reflectivity

The Stimsonite 96, recessed, and Dura-Brite snowplowable markers maintained their reflectivity during the evaluation period, and each of these markers provided excellent delineation. Although the Prismo marker was less reflective than these markers, it maintained its reflectivity and provided satisfactory delineation. The Kingray marker suffered a severe loss of reflectivity. A subjective rating of the reflectivity of these markers revealed that the Stimsonite 96 marker was the best overall. The reflectivity of the recessed marker varied somewhat with roadway geometry, but it could be rated as second. Because the Dura-Brite marker was a lower-profile marker (rising only 0.25 in. above the pavement surface), the test results indicated that it had slightly lower reflectivity, and it was given a subjective rating of third. Nevertheless, the Dura-Brite marker still provided more than adequate delineation, and the low profile of this marker provides some durability advantages. A new Stimsonite marker, which was recently introduced, is also a low-profile marker and will probably be similar to the Dura-Brite in reflectivity. It should also be noted that all of the marker types performed satisfactorily during light snow and ice conditions.

Durability

Considering only traffic wear, the Kingray and Prismo markers were the only markers that experienced any significant damage. The Dura-Brite and recessed markers received the least amount of damage. The Stimsonite 96 sustained minor damage to the lens in a few markers.

Evaluation of the snowplow operations revealed that the Stimsonite 96, Dura-Brite, and recessed markers qualify as snowplowable markers. None of these markers sustained any noticeable damage as a result of the limited number of snowplow operations. The evaluation revealed that the Prismo markers were not snowplowable, and the Kingray markers sustained significant damage as a result of snowplow operations.

Another factor that should be considered is the relative snowplowability of the markers. The concept used in the design of the Stimsonite 96 and the Dura-Brite markers is to retain the reflective unit of a raised pavement marker and attempt to protect it by using a snowplow-resistant encasement. Nevertheless, the tests indicated that an encasement sufficiently sturdy to resist snowplow damage will likely interfere with snowplow operations because of severe vibrations and will probably damage the blade. Of the markers evaluated in this study, only the recessed and Kingray markers would present a sufficiently low profile (or characteristics that cause them to function like a low-profile marker) to not interfere with snowplow operations.

RECOMMENDATION

The Stimsonite 96, Dura-Brite, and recessed markers should be considered as acceptable snowplowable markers. All three had adequate reflectivity, which was maintained during the test period, and proved to be durable when subjected to snowplow operations. Nevertheless, considering all available input, the recessed marker is recommended as the most functional and cost-effective marker. This recommendation is based on the following characteristics of the recessed marker: (a) ease of installation, (b) high retention of reflectivity, (c) durability when subjected to snowplow operations, (d) relative cost of the marker and its installation, and (e) lack of interference with normal snowplow operations. Specifications for an installation contract of snowplowable markers could allow for use of any of these three markers (Stimsonite 96, Dura-Brite, and recessed), but considering available cost data, the recessed marker should provide the lowest cost.

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Evaluation of the Effect of Natural Brine Deicing Agents on Pavement Materials

RONALD W. ECK, MUMTAZ A. USMEN, WILLIAM A. SACK, IRFAN ARAR, AND ALI ATEFI

Laboratory tests were conducted to analyze and compare the effects of natural brines and traditional deicing chemicals on bituminous concrete, portland cement concrete, and two types of steel. The effects of 100 freeze-thaw cycles on bituminous concrete immersed in distilled water, a sodium and calcium chloride solution, and a natural brine solution were evaluated by testing specimens for Marshall stability, flow, and weight changes. Two separate 100-cycle freeze-thaw experiments—a surface degradation test, and compressive strength and pulse velocity tests—were conducted to evaluate the effects of distilled water, a sodium and calcium chloride solution, and a natural brine solution on concrete performance. A 100-cycle wet-dry immersion test, where specimens of A-36 and SAE-1010 steel were subjected to a sodium and calcium chloride solution and a natural brine solution, was used as an accelerated corrosion test. Corrosion was measured by weight loss. In general, the effects of natural brines on bituminous concrete were no different than the effects of traditional deicing agents. In terms of the surface deterioration of portland cement concrete, natural brine performed slightly better than traditional deicing agents. Potential discoloration of concrete from brines with high iron content was indicated. The effect of natural brine on concrete compressive strength was no different from that of water or sodium and calcium chloride deicing agents. Specimens of automobile-body steel demonstrated less corrosion in natural brine than in a sodium and calcium chloride solution; however, specimens of structural steel demonstrated opposite results.

As expenditures for snow- and ice-control materials continue to increase, highway agencies are seeking ways to minimize the use of traditional deicing materials and, where possible, are substituting less-costly deicing agents for sodium and calcium chloride. The use of naturally occurring salt brines, which are by-products of oil and gas production, as deicing agents is either a reality in some locations or is being studied in a number of locations. Natural brines are widespread geographically; their existence is not limited to oil and gas fields. The oil and gas industry considers brine as a waste product because there is no apparent use for the liquid, which may be several times stronger than sea water.

The major ions found in most brines are sodium, calcium, magnesium, and chloride. Lower levels of constituents such as potassium, iron, sulfate, and bicarbonate are also usually present. In addition, brines may contain a number of minor or trace ionic species, including bromide, iodide, barium, lead, arsenic, zinc, cadmium, and chromium.

Brines are difficult to dispose of in an environmentally acceptable manner; thus they represent a major problem for the oil and gas industry. For example, more than 100 million gallons of brine are produced annually in West Virginia. Several large brine producers make use of injection wells for brine disposal. In addition to being an expensive disposal method, there is the possibility that increased concern about groundwater contamination may result in legislation that will limit deep well injection. An unknown but significant quantity of waste brine is discharged directly onto the ground or into surface waters, which results in contamination. Eck and Sack (1) have presented a more detailed discussion of brine characteristics and current disposal practices.

The use of natural brines for deicing purposes would appear to solve several problems simultaneously. The oil and gas industry could dispose of an unwanted by-product, and highway agencies could acquire a deicing material at minimal cost. However, before advocating a major deicing program that uses natural brines, a number of issues need to be evaluated. For example, the quantity of brine available for highway deicing in a given geographical area must be assessed. Brine quality from the major producing formations must be determined, including both the major salts and the minor trace elements, in order to assess potential water pollution problems. It would be desirable to compare various brines with commercial deicing agents relative to melting, skid resistance, and refreezing of roadway surfaces. Also, the transportation and storage costs of brine must be estimated. Similarly, comparisons are needed as to the relative effect of brines and commercial deicing agents with respect to corrosion of steels as well as the deterioration characteristics of portland cement concrete (PCC) and bituminous pavement.

A comprehensive research project is in progress at West Virginia University (WVU) to address these

Table 1. Chemical analysis of brine from Chemung Formation, Lewis County, West Virginia.

Item	Value
Ions (mg/L)	
Total dissolved solids (TDS)	215 240
Sodium	57 780
Calcium	23 010
Magnesium	3 116
Potassium	450
Chloride	136 143
Sulfate	198
Iron	210
Barium	4.85
Lead	3.86
Cadmium	0.869
Zinc	0.680
Arsenic	0.231
Chromium	<0.050
pH	4.00

questions. The study includes both laboratory and field testing programs. The procedures used and the results obtained in a laboratory evaluation of the effects of natural brines on three common transportation materials--bituminous concrete, PCC, and steel--are presented in this paper. Other aspects of the project, although important in assessing the overall feasibility of natural brines as deicing agents, are beyond the scope of this paper. Results of a field testing program, where brine was applied to actual roadways, are described elsewhere (1).

BACKGROUND AND OBJECTIVES

There is a wealth of information in the literature describing the effects of sodium and calcium chlorides on transportation materials. The detrimental effects of commercial sodium and calcium chloride deicing agents on PCC and steel have been well documented in many studies. Several recent surveys of these efforts are available in the literature (2-4). The effect of these agents on bituminous concrete has received less study. This is probably because the few studies that have been performed (5,6) indicated that deicing salts had no significant effect on bituminous concrete mixes. Because the large number of constituents in natural brines makes them different from commercial deicing agents, previous research findings about the effects of commercial agents on transportation materials are not necessarily transferable to natural brines.

The effects of deicing agents on the corrosion of metals have received much attention. Two recent reviews (2,3) have assessed the effects of deicing chemicals on metals. It is concluded that chloride deicers accelerate the corrosion of motor vehicles and metal highway structures.

The literature dealing with natural brine as a deicing agent is limited. Although a number of state, county, and local highway agencies have used or are using natural brines (7-9) as deicing agents, they have not, in general, evaluated or monitored the impacts associated with brine use. The environmental aspects of using Arkansas brine for highway purposes have been examined (10), although the study did not include a field evaluation. Laboratory studies were used to determine the effect of brine on concrete, metal corrosion, soil properties, and runoff water quality. The study concluded that the effects of Arkansas brine on metal corrosion rates and concrete strength were not significantly different from those of an equivalent sodium chloride solution.

No published literature could be located relative to the effects of brines from Appalachian oil and gas fields on transportation materials. Because brine composition varies from one location to another, research with different brines appeared warranted. Thus the overall goal of the research described herein was to analyze and compare the effects of West Virginia brines and traditional deicing chemicals on bituminous concrete, PCC, and two types of steel.

EXPERIMENTAL PROCEDURES

Brine Characteristics

The brine used was taken from the Chemung Formation in Lewis County, West Virginia. A chemical analysis of the brine is given in Table 1; the analysis reveals that the major constituent ions are sodium, calcium, magnesium, and chloride. Lower levels of ions such as sulfate, potassium, and iron are also present, along with trace-level concentrations of heavy metals such as barium, lead, and zinc. Sulfate levels are of particular interest with respect to the attack of concrete. Sulfate levels in brine vary widely but are usually less than 700 mg/L. For example, an analysis of nine West Virginia brines revealed an average sulfate concentration of 126 mg/L, with a range of 3 to 547 mg/L. In a recent study Pierce (11) concluded that sulfate levels of 150 mg/L or less would result in a negligible attack on concrete, whereas levels of 150 to 1500 mg/L would cause a positive degree of attack, and above 1500 mg/L the degree of attack would be severe. Based on these considerations, the sulfate level in most brines would not be expected to contribute significantly to concrete deterioration.

Trace metal levels in certain brines may be of environmental concern. In the brine used in this study, lead and barium were present at concentrations of 4 to 5 mg/L, whereas other trace elements were measured at less than 1 mg/L. Although a complete discussion of this issue is beyond the scope of this paper, it should be noted that conventional dry deicing agents analyzed as part of the overall project at WVU were also found to contain trace metals at levels comparable to those found in brine. It should also be noted that, when considering the potential impacts of constituents such as trace metals or sulfate, considerable dilution of the brines will occur during melting and runoff after application.

Bituminous Concrete

In designing the bituminous concrete mixture, an aggregate gradation and an asphalt cement representative of those used in wearing courses in West Virginia were used. Asphalt Institute Mix Type IVb, which is dense graded and has a maximum nominal size of 0.5 in., was chosen. Materials were acquired from local sources that furnish asphalt mixes for West Virginia Department of Highways (WVDOH) projects. An AC-20 viscosity graded asphalt cement, which had a specific gravity of 1.03 and conformed to the requirements of AASHTO M-226, was used. The aggregates--limestone sand and No. 67 coarse limestone aggregate--were obtained from the same supplier.

The Marshall method (12) of mix-design performed in the study encompassed the fabrication of replicate specimens at varying asphalt contents by using 50 blows of drop hammer compaction and subjecting these specimens to bulk specific gravity, Marshall stability, and flow tests as well as a density-voids analysis. An optimum asphalt content of 6.2 percent was established by using the Marshall mix-design criteria. The mix, as designed, had an average Marshall stability of 2,500 lb, a flow value of 0.18 in., a unit weight of 147.5 lb/ft^3, 3 percent air voids, and 15 percent voids in mineral aggregate (VMA). Note that the flow value, although somewhat high, was still in the acceptable range.

The bituminous mixture just described was used in the preparation of 50 standard Marshall specimens. These specimens were divided into a control group and three treatment groups (immersion in distilled water, a sodium and calcium chloride solution, and a natural brine solution), and each was divided into three subgroups (exposure to 10, 50, and 100 freeze-thaw cycles), such that a total of 10 groupings of 5 replicate specimens each were generated (50 specimens overall). Groupings were established such that the difference between the mean bulk specific gravity of each group was less than 0.010.

Group 1 specimens were for control purposes and received no treatment in the form of immersion or exposure to freezing and thawing. Specimens in group 2 were immersed in distilled water and were subjected to the specified number of freeze-thaw cycles. Group 3 specimens were immersed in a 100 000 mg/L total dissolved solids (TDS) solution of sodium and calcium chloride, which had a 50:1 ratio of sodium to calcium. This solution was prepared by dissolving in distilled water carefully weighed quantities of dry sodium chloride and calcium chloride that were obtained from WVDOH stockpiles. Although the two salts are no longer normally used in combination in West Virginia, the rationale behind choosing this solution was to devise a possible worst case that could occur on a roadway or bridge treated with both salts at different times.

Specimens in group 4 were immersed in a 100 000 mg/L TDS solution of natural brine that had been diluted with distilled water. The original concentration (TDS) of this brine was 215 240 mg/L, as noted in Table 1. It was diluted to achieve a concentration equivalent to that of the sodium and calcium chloride solution used in the the treatment of the group 3 specimens. This was necessary for drawing meaningful comparisons between results obtained from the two groups. The TDS of 100 000 mg/L was selected for use in the study because it appeared to be a convenient midrange of deicing-agent concentrations found in a comprehensive review of previous research and practice. However, it should be noted that the brine applied to roadways would be used straight and not prediluted. Diluted brine was applied to bare concrete to account for the fact that, in practice, deicing agents are diluted by the accumulation of snow and ice on the pavement.

The specimens were placed in cylindrical 5-in.-diameter and 6-in.-high containers that were fabricated by wrapping several layers of heavy-duty aluminum foil around a hardware cloth (0.25-in. mesh) frame and sealing the base with paraffin. These containers had been previously used in related research at WVU and provided a flexible, yet watertight container capable of holding sufficient quantities of solutions to cover the specimens. The containers with the specimens were then placed in a programmable environmental chamber.

Temperature limits for the freeze-thaw tests were 0° and 40°F. Each cycle consisted of 2 hr of exposure at 40°F, followed by a gradual decrease in temperature to 0°F during a 2-hr period; exposure at this temperature for another 2 hr; and finally a gradual increase in temperature to 40°F during a 2-hr period, for a total of 8 hr. With the aid of the automated chamber, three such cycles could be accommodated per calendar day. The specimens remained in the chamber during the entire investigation; daily observations were made during this period.

The effect of different treatments on the performance characteristics of bituminous concrete was evaluated by testing the grouped specimens for Marshall stability, flow, and weight changes. The specimens in group 1 were weighed and subjected to Marshall stability and flow tests (ASTM D-1559) immediately after fabrication. The initial weights of all other specimens were also recorded before treatments. When the appropriate number of cycles in each treatment (10, 50, or 100 cycles) were reached, specimens were removed from the environmental chamber and allowed to warm to room temperature. Critical visual observations were then performed on the specimens. Finally, the specimens were photographed, weighed, and tested for Marshall stability and flow.

Portland Cement Concrete

Two separate tests were conducted to evaluate the performance of concrete: (a) a surface degradation test and (b) compressive strength and pulse velocity tests. Both tests were conducted on specimens prepared from a PCC mix typical of that used for highway pavements in West Virginia. The concrete mix data indicated a water-cement ratio of 0.41; fine aggregate made up 41 percent of total aggregates. Enough Daravair air-entraining agent was added to give average strength. The mix yielded a slump of 1.75 in., and the air content was 8.75 percent.

Specimens for each test were allowed to cure at room temperature for 24 hr under a plastic cover. The molds were then removed and specimens were moist-cured (100 percent relative humidity) for 14 days at an average temperature of 70°F. They were then air-cured at 73°F for another 14 days.

The solutions of natural brine and traditional deicing agents used in the test were identical to those used in the bituminous concrete experiment. Characteristics of the solutions were presented in the preceding section.

Surface Degradation Tests

Test specimens were divided into five groups. Specimens in group 1 were subjected to freezing and thawing in distilled water. Those in group 2 underwent freezing and thawing with a sodium and calcium chloride solution. Group 3 specimens were subjected to freezing and thawing with natural brine. Groups 4 and 5 underwent wet-dry cycles while exposed to sodium and calcium chloride and natural brine solutions, respectively.

Two different procedures, freeze-thaw and wet-dry, were selected to simulate field conditions existing when deicing chemicals are applied. Five specimens were fabricated for each group. ASTM C-672 (Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Salts) was followed.

A 10-in.-diameter polyvinyl chloride (PVC) plastic sewer pipe was cut into 3.15-in. sections to serve as molds for the concrete. Each section was then slit to ensure ease of removal of the cured specimen. Wire was wrapped around the perimeters of the pipe sections to keep them from opening during placement of the concrete. These pipe sections were then clamped to sheets of plywood. Concrete was placed and finished in accordance with ASTM C-672. As specified, specimens were surface-roughened by brushing them with a medium stiff brush to simulate the finish on a highway pavement.

The pipe sections, used previously as molds, were retained for use as dikes. This was accomplished by slipping the pipe sections over the cured specimens with about 1 in. protruding above the concrete surface as a dike. Wire was wrapped around the pipe section to hold it secure during the test. Caulk was applied to the crack formed by the pipe around the perimeter of the specimen to prevent leakage of the solution.

Freeze-thaw concrete specimens were alternately stored in two small temperature-controlled rooms. One room had a nominal temperature of 0°F, and the other was at 50°F. Because the rooms were at constant temperature, it was necessary to move the specimens twice daily (by using a cart) between the rooms to produce the freeze-thaw cycles.

Specimen groups 1, 2, and 3 were subjected to daily cycles of freezing at 0°F for 16 hr and thawing at 50°F for 8 hr, with 0.25 in. of the appropriate solution covering the surface of each specimen. All specimens were kept in the freezer during weekends. As evaporation occurred, distilled water was added to maintain a 0.25-in. depth of the solution. The testing period for these groups was 100 cycles.

Specimen groups 4 and 5 were subjected to wet-dry cycles. In the wet portion of the cycles each specimen was covered with 0.25 in. of the appropriate deicing solution for 24 hr. The dry phase of the cycle started after the salt solution was washed from the specimens with distilled water; specimens were then allowed to dry for 24 hr. Therefore, for groups 4 and 5, one complete cycle required 48 hr. The entire cycle was performed in the laboratory at essentially constant room temperature (about 72°F). The test continued for 100 cycles.

At the end of every five cycles the surfaces of all specimens were rinsed with distilled water, visually inspected, and rated for scaling by using the 0 to 5 subjective rating scale specified in ASTM C-672. Photographs were taken of specimen surfaces after every five cycles.

Compressive Strength and Pulse Velocity Tests

Specimens were divided into four groups, with each group consisting of four cylindrical (4 x 8 in.) specimens. After curing the cylinders were placed in plastic containers with sufficient solution to cover them completely. Specimens in group 1 were tested for pulse velocity and compressive strength immediately after curing. Group 2 specimens underwent freezing and thawing while completely submerged in distilled water, whereas specimens in group 3 were exposed to freezing and thawing while completely submerged in a sodium and calcium chloride solution. Specimens in group 4 were subjected to freezing and thawing while completely submerged in natural brine. The pulse velocity test was used in this study to obtain periodic evaluations of concrete quality.

Cylinders in groups 2, 3, and 4 were tested for pulse velocity while completely submerged in appropriate solutions, and then were subjected to daily cycles of freezing at 0°F for 16 hr and thawing at 50°F for 8 hr. Tests were conducted in the same chambers described previously for the surface degradation tests. At the end of every five cycles the cylinders were wiped dry with a towel and the pulse velocity was measured and recorded for each specimen by using procedures outlined in ASTM C-597. Specimens were then resubmerged in the appropriate solution and subjected to additional freeze-thaw cycles. Both pulse velocity and compressive strength were measured at the end of 100 cycles.

Steel Corrosion

A wet-dry immersion test, where metal specimens were alternately immersed and removed from salt solutions, was used in this study as an accelerated corrosion test. Corrosion was measured by weight loss. The corrosion test essentially followed ASTM G-31 (Laboratory Immersion Corrosion Testing of Metals) and G-1 (Preparing, Cleaning, and Evaluating Corrosion Test Specimens).

Two types of steel were tested: A-36 steel was chosen to represent the steel used in highway construction and SAE-1010 steel was selected as being representative of that used in automobile bodies. Coupon specimens 2 in. wide by 4 in. long were used. A-36 steel specimens of these dimensions had been cut from 2-in.-wide by 0.25-in.-thick hot-rolled bar stock by a shear blade at the factory. Although it would have been desirable to use a thinner gauge for testing purposes, the 0.25-in. thickness was the minimum size available locally. The SAE steel was obtained in 4-in.-wide by 12-in.-long sheets of 24 gauge (nominal thickness 0.0239-in.) cold-rolled steel. By using a shear blade in the laboratory, the sheets were cut into 2 x 4 in. specimens. Forty-eight specimens from each of the two types of steel were used in the test. With two different solutions and six time periods, this meant there were four replications for each test condition.

Four 50-L plaster picnic coolers were used to hold the test solutions and metal coupons. In addition to having the desired capacity, the coolers had attached lids that made convenient covers to keep out dust and other contaminants. A drain in the bottom made it possible to expose the coupons to air simply by lowering the solution level rather than by lifting the specimens out of the solution and disturbing them. The solutions drained from the coolers were stored in plastic bottles. Coolers were refilled by placing the plastic bottles at a higher level than the coolers and letting the solutions flow back to the coolers. Wooden spring-loaded clothespins were placed over glass rods positioned across the top of the cooler; the metal specimens were suspended from the clothespins into the solutions. The sodium and calcium chloride solutions used in the corrosion test were identical to those used in the bituminous concrete and PCC tests described earlier.

Specimens were subjected to daily cycles of immersion for 16 hr and drying for 8 hr. Both immersion and drying took place in the laboatory at room temperature (about 73°F). During the wet phase of the cycle specimens were immersed completely in the appropriate solution. The dry phase was achieved by lowering the solution level 0.5 in. below the bottom edge of the specimens. At the end of the 8-hr dry phase the coolers were refilled with the same solutions. On weekends the specimens remained in the immersed state. Every 2 calender weeks the solutions in the coolers were replaced with fresh solution of the same concentration as the original.

Specimens were prepared in 4 groups of 24 specimens each. The coupons were cleaned according to ASTM G-1. The A-36 steel specimens were wet sanded under running distilled water; this step was not necessary for the SAE steel coupons. All specimens were rinsed thoroughly in running distilled water and scrubbed with a stiff fiber-bristle brush using a bleach-free scouring powder. Specimens were then pickled in an acid bath, rinsed in distilled water, and placed in an acetone bath. Finally, the specimens were weighed and photographed before being placed in the appropriate cooler.

After the designated number of wet-dry cycles, the appropriate group of specimens was removed, photographed, and visually evaluated. They were then cleaned according to ASTM G-1. Loose corrosion products were removed by carefully scraping the specimen with a razor blade. Any stubborn corrosion was removed with a fiber-bristle brush. Specimens were dipped in a pickling bath, the solution was stirred vigorously, and specimens were rubbed with a nonabrasive rubber implement. Specimens were rinsed with distilled water and bathed in acetone before being weighed and photographed.

RESULTS

Bituminous Concrete

Results obtained for the bituminous concrete performance characteristics studied are shown in Figures 1-3. The mean Marshall stabilities presented in Figure 1 for bituminous concrete specimens tested immediately and then after 10, 50, and 100 freeze-thaw cycles indicate that the specimens exposed to distilled water all experienced a reduction in Marshall stability, whereas all but one of the specimens exposed to salt solutions had higher stabilities than the control value. The differences between the specimens subjected to rock salt and natural brine solutions were not significant. The results obtained from Marshall stability tests after 100 freeze-thaw cycles in the laboratory do not indicate a serious degradation of specimens on exposure to any of the solutions.

The mean flow values shown in Figure 2 for specimens tested immediately after curing and then after 10, 50, and 100 cycles reveal that, with the exception of those subjected to 10 cycles of freezing and thawing in rock salt solution, all specimens pro-

Figure 1. Marshall stability test results for bituminous concrete specimens subjected to freeze-thaw cycles of water, rock salt solution, and natural brine.

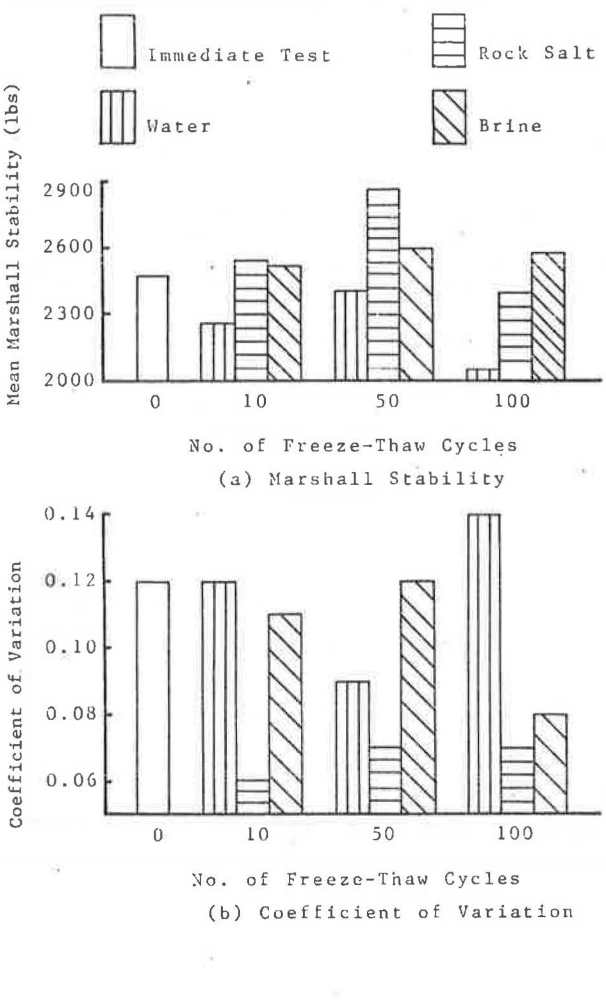


Figure 2. Marshall flow test results for bituminous concrete specimens subjected to freeze-thaw cycles of water, rock salt solution, and natural brine.

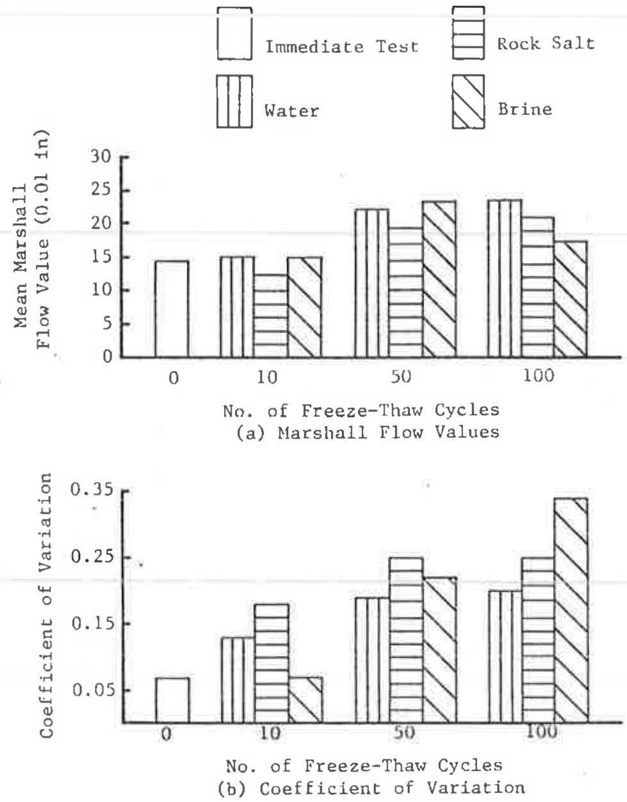


Figure 3. Water change test results for bituminous concrete specimens subjected to freeze-thaw cycles of water, rock salt solution, and natural brine.

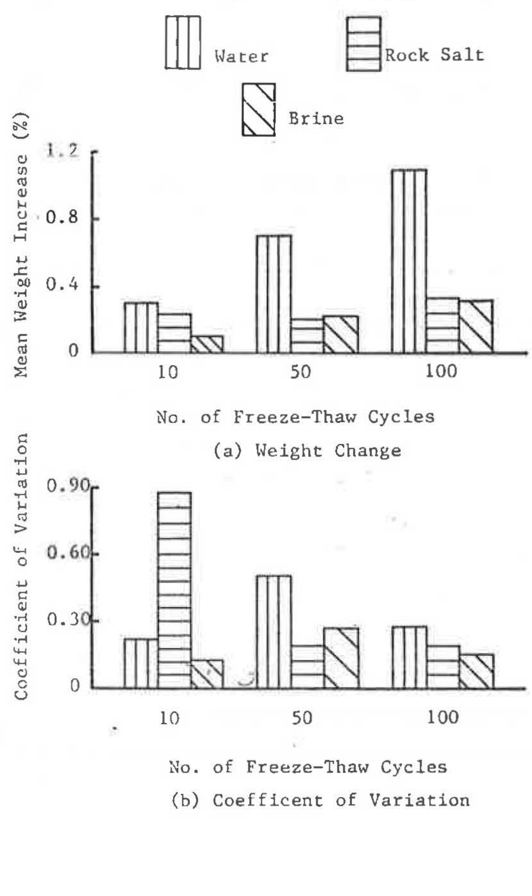
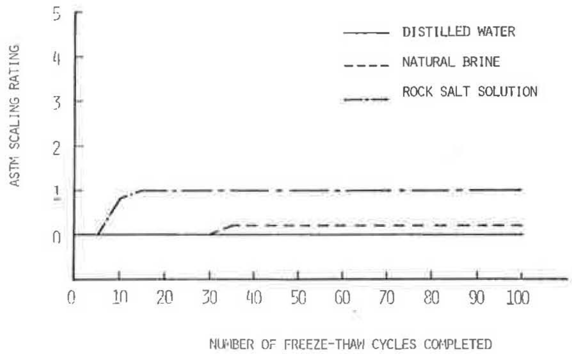


Figure 4. Effect of deicing-agent solution on surface scaling of specimens subjected to freeze-thaw cycles.



duced flows higher than the control group. Because the mixture had a high flow value as designed, this is not believed to be of great concern. It has been suggested in a previous study (5) that increases in Marshall flow values would be expected because of volume increases in specimens or because of severe deterioration. None of these causes was discernible in the specimens tested in this study. Furthermore, the ranges (difference between extreme measurements) and coefficients of variation of flow values were relatively high, which indicated possible testing variability.

Any significant loss of aggregate through scaling or raveling on exposure to any of the test solutions would be indicated by a specimen weight loss. Weight changes of surface-dry specimens, which are expressed as a percentage of original specimen weight, are shown in Figure 3. All specimens gained weight during the testing; there was no loss of aggregate because of specimen exposure to the test solutions. The small weight gains can be attributed to normal increase in moisture content of specimens exposed to the solutions and are not considered detrimental to performance.

Specimens subjected to the rock salt solution and to natural brine experienced a smaller weight increase than specimens subjected to distilled water. This is attributable to increased density and viscosity of the solutions because of the dissolved salts. There was essentially no difference in weight change between those specimens exposed to rock salt solution and those exposed to natural brine.

In addition to the quantitative tests just described, specimens were photographed and visual observations were made both before and after testing. No loose particles or losses of aggregate were noted when specimens were removed from the solutions. There was little, if any, difference in appearance between the original specimens and those subjected to 100 test cycles in the three solutions.

Portland Cement Concrete

Surface Degradation Tests

The effect of each of the three solutions on the scaling resistance of PCC subjected to freezing and thawing is shown in Figure 4. Specimens covered with distilled water showed no change in surface condition; the visual rating of the surface remained 0 (no scaling) during the entire 100-cycle study period for all five specimens.

Concrete specimens subjected to the sodium and calcium chloride solution showed the earliest scaling. After the 10th cycle, four of the five specimens were rated at a value of 1. The remaining specimen received a rating of 1 at the 15th cycle. The visual rating remained at this value for all specimens for the duration of the test period.

Concrete specimens subjected to natural brine solution scaled later and showed less scaling than specimens subjected to the sodium and calcium chloride solution. Only one specimen was rated 1 after 35 cycles; the other four specimens did not display scaling during the 100 cycles of the test. Thus natural brine appears to be no worse than traditional deicers in terms of detrimental effects on PCC pavement surfaces.

Specimen groups 4 and 5 underwent wet-dry cycles and were rated by using the same scheme as that for the freeze-thaw tests. Neither the specimens subjected to brine nor those exposed to the sodium and calcium chloride solution showed any scaling through the 100 cycles. Nevertheless, as the cycling continued the surfaces of the specimens subjected to brine took on an orange-brown rust color, apparently because of the iron in the brine. This discoloration became more conspicuous as the number of cycles increased. Scratching the surface with a metal spatula indicated that this was a surface phenomenon. Thus, although the color may be undesirable from an aesthetic standpoint, it does not appear to affect performance. It must be noted that the testing performed here was a severe test because fresh brine was applied to the concrete surface every 48 hr and allowed to stand for 24 hr. On an actual pavement brine would be removed from the surface by running water and by the sweeping action of vehicle tires; thus actual discoloration is expected to be insignificant. It is possible that the discoloration tendency of brine solutions may affect pavement markings after prolonged use. Application of brine, over a period of time, to an actual section of PCC pavement is recommended to determine whether discoloration will be a problem.

Table 2. Summary of results of concrete compressive strength and pulse velocity tests.

Test Condition	Mean Compressive Strength (psi)	Mean Final Pulse Velocity (ft/sec)
Control	3,902	13,962
Water	3,294	13,852
Sodium and calcium chloride	3,318	13,858
Natural brine	3,542	13,851

Compressive Strength and Pulse Velocity Tests

The mean compressive strength values for the control group and for cylinders subjected to 100 freeze-thaw cycles are given in Table 2. Based on the data in the table it appears that the effect of natural brine on concrete compressive strength is no worse than distilled water or sodium and calcium chloride solutions. Actually, the cylinders immersed in natural brine solution had the smallest reduction in compressive strength, compared with the control specimens, of any of the test solutions.

Mean final pulse velocities for the control group (tested immediately after curing) and for the cylinders subjected to distilled water, the sodium and calcium chloride solution, and natural brine are also given in Table 2. Final pulse velocity values for the three groups were nearly identical and only slightly less than those of the control group. Pulse velocity measurements for the groups did not indicate any significant difference when plotted versus time. Based on the results of the pulse

Figure 5. Relation between weight loss per surface area and number of cycles completed for SAE steel specimens.

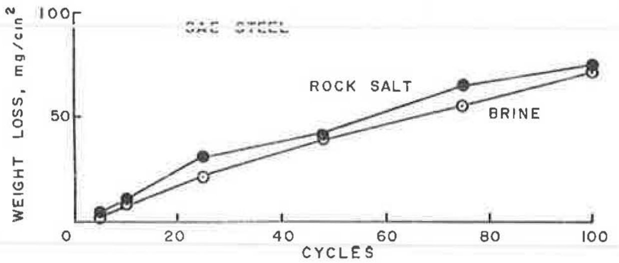
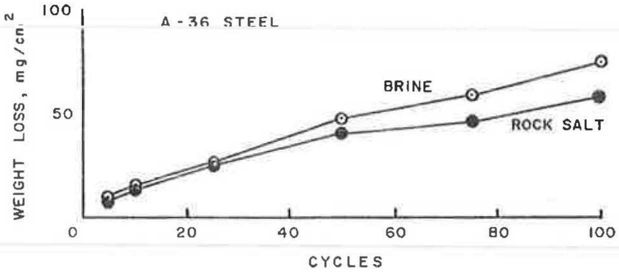


Figure 6. Relation between weight loss per surface area and number of cycles completed for A-36 steel specimens.



velocity test given in Table 2, it can be concluded that, although the freeze-thaw cycles had a slightly detrimental effect on concrete performance, the natural brine solution was no more harmful than the sodium and calcium chloride solution. Furthermore, the salt solutions caused no more degradation than did the distilled water.

Steel Corrosion

SAE Steel

Weight loss data for the automobile-body steel specimens are shown in Figure 5. As expected, the curves appear to be becoming asymptotic. Specimens immersed in the sodium and calcium chloride solution lost more weight than those immersed in natural brine (4 percent difference after 100 cycles).

During the cleaning process it was noted that corrosion products were easier to remove (with a razor blade) from the specimens immersed in brine than for those immersed in the sodium and calcium chloride solution. This was attributable to the smoother surface on the brine specimens. The coating of corrosion products was equally distributed over the surface of the specimens immersed in natural brine after 25 cycles. However, for the specimens immersed in the sodium and calcium chloride solution for the same period of time, the coating was thicker around the edges of the specimen than in the interior. Both of these characteristics persisted throughout the remainder of the test. Whiskers (or spikes) were noted on the surface of the specimens immersed in the sodium and calcium chloride solution after about 75 cycles. The whiskers were about 0.125 in. long and approximately 0.25 to 0.125 in. in diameter; density of the whiskers was about 0.3 whiskers per square centimeter.

As specimens were removed from the solution, a consistent pattern became apparent. Surfaces of specimens immersed in the sodium and calcium chloride solution were noticeably rougher; they had both high and low areas. After 75 and 100 cycles the bottom edges of specimens immersed in the sodium and calcium chloride solution were severely corroded and lost about 0.2 cm from their original length. Similarly, the other three edges had lost their original shape. In contrast, weight loss for specimens immersed in natural brine was uniformly distributed over the entire specimen. It is believed that one or more of the constituents of the brine inhibited corrosion and led to a more uniform weight loss. Careful examination of the specimens revealed no pitting in any of the groups, even after 100 wet-dry cycles. Because there are no large crystals to become trapped against the surface of the metal, liquid deicing agents would be expected to be less of a problem as far as pitting corrosion is concerned.

A-36 Steel

Weight loss results for the structural steel specimens are shown in Figure 6. The specimens immersed in both solutions had essentially identical weight losses through the 25th cycle. Beyond this point the specimens immersed in natural brine lost more weight than those in the sodium and calcium chloride solution, with a difference of 22 percent after 100 cycles.

The alternate immersion of the A-36 steel specimens had different results from the SAE steel. For specimens in both solutions, the coating was thicker around the edges than in the interior. However, the coating on the surface of the specimens immersed in the sodium and calcium chloride solution had high and low areas as opposed to the smoother surface of the specimens immersed in brine. As with the automobile-body steel, it was easier to remove the corrosion products from the surface of the specimens immersed in brine than from those in the sodium and calcium chloride solution. After 50 cycles whiskers appeared on the surfaces of specimens immersed in the sodium and calcium chloride solution; this phenomenon was not evident on the specimens immersed in natural brine. These whiskers were slightly longer than those described previously, but otherwise they had similar characteristics.

Surface characteristics of the A-36 steel were different from those that had been obtained with the SAE steel. Structural steel specimens in both solutions exhibited rough surfaces with high and low areas. There was no pitting in any of the A-36 specimens after 100 cycles. Some specimens displayed a tendency toward cratering. As defined in this study, a crater is a circular depression whose diameter is greater than its depth. It is possible that these characteristics are caused by different stresses in the surface from the rolling process. Stressed areas would be expected to corrode more readily than unstressed areas.

As has just been discussed, some of the results from the A-36 steel were different from those obtained with the SAE steel. The most likely explanation for this is the different chemical composition of the A-36 steel. The duration of the tests on the A-36 steel may have been too short. Some of the phenomena described would be more emphasized over time, and a longer test would provide a better explanation of the processes involved.

CONCLUSIONS

Based on the results of 100 freeze-thaw cycles in the laboratory, the effects of natural brines on bituminous concrete pavements appear to be no different than the effects of traditional rock salt deicing agents. Thus, from a materials standpoint, there appears to be no reason why natural brines could not be used as a deicing agent on bituminous concrete pavements.

In terms of PCC surface deterioration after exposure to 100 freeze-thaw cycles in the laboratory, natural brine performed somewhat better than traditional chloride deicing agents. Similarly, it can be concluded that the effect of natural brine on concrete compressive strength is no different from that of water or sodium and calcium chloride deicing agents. Although some discoloration of the concrete occurred because of iron in the brine, the problem is not expected to be significant under actual field conditions. Further study is warranted.

No general statement can be made about the effects of natural brine on the corrosion of steel. The effects will depend on the chemical composition of the steels and the constituents of the brine. In this study, which involved 100 wet-dry cycles, specimens of automobile-body steel demonstrated less corrosion in natural brine than in a sodium and calcium chloride solution. However, specimens of structural steel demonstrated opposite results.

RECOMMENDATIONS

Additional consideration should be given to the composition of natural brines. Although the sulfate levels in the brine used did not appear to present problems as far as concrete deterioration, it would be desirable to examine other brines with higher sulfate levels. In addition, the duration of the test should be longer than that used in this study. Trace metal levels in certain brines may be of environmental concern; therefore, there is a need for additional work in this area. Because brine use is expanding beyond deicing applications to the area of dust control, such additional study is important.

The results of the research described in this paper suggest several other areas for further study. It would be desirable to investigate the effects of natural brines on bituminous concrete, PCC, and various types of steels in the field. For PCC, field testing should focus on the potential discoloration caused by natural brines. For steel, major emphasis should be placed on evaluating the long-term corrosive effects of natural brines. It would also be desirable to test a reinforcing-type steel in concrete to provide an indication of corrosion behavior under these conditions. In all cases field studies should be supplemented by additional laboratory tests of longer duration than those performed in this study. The 100-cycle testing used may not have been sufficient to provide an indication of the long-term effects of some brine constituents.

The study presented here used phenomenological approaches in studying the three materials. The physical and chemical behavior responsible for the results observed was not examined. Additional examination of these phenomena at the microscopic level would be beneficial.

The natural brine evaluated in this study is typical of that found in the northern and central Appalachian region. As noted earlier, however, brine composition may vary significantly with depth of occurrence and the formation in which it is located. Additional experimentation is warranted by using other brines that contain different ratios of major and minor constituents.

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