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COVER: Wide edgelines may reduce accidents on curves.

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Edgeline Widths and Traffic Accidents

by Harry S. Lum and Warren E. Hughes

Introduction

Since the 1950's, the use of edgelines has become an accepted practice in the United States—especially on roads carrying high volume. The normal edgeline width is between 4 in (10 cm) and 6 in (15 cm). Some States, however, are now using the wider 8-in (20-cm) edgeline in special places such as freeway gore areas. Kansas and Ohio were probably the first States to research the safety benefits of edgelines. In 1956-57, Ohio conducted a before and after study using 116 mi (187 km) of two-lane rural highways for control and treatment sites. It concluded that the after period with 4-in (10-cm) edgelines significantly reduced the number of accidents as compared with those sites with no edgelines. (*1*)[1] In a similar before and after study in 1959-60 Kansas concluded that the 4-in (10-cm) edgelines showed a significant reduction in the number of accidents at access points (i.e., driveways and intersections.)(*2*) The Kansas study involved control and treatment sites comprising 384 mi (618 km) of rural highway servicing between 550 and 3,600 vehicles per day.

More recently, New Mexico and Virginia conducted studies to evaluate the effectiveness of the wide (8-in [20-cm]) edgelines in preventing run-off-the-road accidents. (*3,4,5,6*) They concluded that the wider edgelines have no significant effect on traffic safety. However, because the number of miles sampled in their studies was small, researchers are hesitant to accept their findings.

In the early 1980's, two research studies were conducted to evaluate drivers' traffic performance in response to different edgeline widths. A 1980 study, conducted in West Milford Township, New Jersey, evaluated the effects of 4-, 6-, and 8-in (10-, 15-, and

[1]Italic numbers in parentheses identify references on page 158.

20-cm) wide edgelines on the performance of 16 male subjects ages 21 to 25. (*7,8*) The subjects drove on an isolated and controlled 24-ft (7.3-m) two-lane road used as a test course between midnight and 3 a.m. Good driving was defined as the ability to remain in the center of the lane. Overall, the subjects performed better—weaving less—on 10 curved sections of the test course using 8-in (20-cm) edgelines than with 4-in (10-cm) edgelines. Further analysis showed that subjects dosed to a 0.05 or 0.08 blood alcohol content (BAC) level exhibited more "good" driving traits on the wider edgeline course. The range was computed for each subject.

The second study, conducted in 1982 at the General Motors-Holden Proving Ground, Lang Lang, Australia, evaluated the effect of different delineation treatments on the performance of 36 male subjects driving on a closed test route with horizontal curves and tangents. (*9*) For the 12 curves analyzed, edgelines added little improvement to drivers' lateral positioning of their vehicles over long-range delineations (e.g., chevrons, post-mounted delineators). Subject-drivers, however, with a BAC of 0.05, showed significantly better lateral positioning of their vehicles at the critical mid-curve point.

In both studies subjects drove through a controlled test course with no following or oncoming vehicle. Subjects were accompanied by an experimenter-driver who would take over in an emergency.

The purpose of the present study was to determine whether the more expensive wide edgelines are cost beneficial in terms of reducing edgeline-related accidents. An edgeline-related accident is defined as a run-off-the-road accident in which the driver is not *forced* off the road—for instance, an accident occurring because the vehicle has drifted out of its lane and off the road. Icy or snow-covered roads were excluded from the analysis since the edgelines cannot be seen.

Also in conjunction with this study, a separate and limited evaluation was conducted to determine the effects of the 4-in (10-cm) and 8-in (20-cm) edgelines on drivers' performance in an uncontrolled environment. Within the curve, the measure of performance was the frequency of encroachment (i.e., tires touching or going over) either the centerline, edgeline, or both.

Data

Seven States participated in this study by furnishing control and test sites for edgeline striping and supplying the necessary traffic, geometric, and accident data. Table 1 shows the sample mileage by State and volume category; table 2 shows—also by State and volume category—exposure in terms of vehicle-miles of travel.

For each State, a 2-year before and after experimental design was developed. Sections were designated as either "control" or "treatment"; in this way two homogeneous groups of sections were defined with respect to traffic volume, edgeline-related accidents, lane and shoulder widths, and frequency of horizontal curves, within the limits of the available data. Control sections were striped with 4-in (10-cm) edgelines while the treatment sections were striped with 8-in (20-cm) edgelines. Because of construction activity, some road sections originally designated for inclusion in the study plan were not restriped; these were deleted from the analysis. Consequently, the final mileage for the control and treatment groups was not equal.

Table 1.—Sample mileage by State and volume category

Test section mileage

State	2,000 < ADT < 5,000 Control group	2,000 < ADT < 5,000 Treatment group	5,000 < ADT < 10,000 Control group	5,000 < ADT < 10,000 Treatment group	Total
Alabama	294	281	0	0	575
Maine	95	97	22	8	222
Massachusetts	127	102	41	78	348
New Mexico	47	44	0	0	91
Ohio	230	171	98	92	591
South Dakota	25	25	0	0	50
Texas	124	120	27	17	288
Total	942	840	188	195	2,165

1 in = 2.54 cm
1 mi = 1.61 km

Table 2.—Sample exposure by State and volume category

Average daily traffic	State	Control group Before	Control group After	Treatment group Before	Treatment group After
		Exposure, in million vehicle miles traveled			
2,000-5,000 vehicles per day	Alabama	979	742	921	794
	Maine	292	274	272	264
	Massachusetts	314	147	314	147
	New Mexico	310	207	150	84
	Ohio	949	673	736	545
	South Dakota	64	52	66	54
	Texas	315	319	276	298
	SUBTOTAL	3,223	2,414	2,735	2,186
5,000-10,000 vehicles per day	Maine	119	131	70	69
	Massachusetts	415	194	564	264
	Ohio	813	559	759	596
	Texas	129	145	76	95
	SUBTOTAL	1,476	1,029	1,469	1,024
	GRAND TOTAL	4,699	3,443	4,204	3,210

1 mi = 1.61 km

While it was highly desirable for the experiment to start at about the same time in all the States, this was not possible. For example, at the time the study was initiated, Ohio and Texas had already begun a program to evaluate the 8-in (20-cm) edgelines in terms of their safety benefits. They agreed, however, to provide their data for this study. New Mexico, which was just getting its own evaluation study of the wide edgeline under way, also agreed to participate in the study by furnishing the required data. Also, because of delays in awarding contracts for striping or road maintenance scheduling problems, 2 years of before and after data were not always available. For these reasons, analysis was performed on a State-by-State basis.

Analysis

Summaries of the accident analyses are shown by State in table 3 for traffic volumes less than 5,000 average daily traffic (ADT) and in table 4 for traffic volumes greater than 5,000 but less than 10,000 ADT.[2] As shown in these tables, the total number of edgeline-related accidents varied greatly among the States and between the control and treatment groups because of unequal sample size, mileage, and time. None of the changes, however, were statistically significant at the 10-percent level.

[2]The Massachusetts data should be viewed with caution. Although the study was focused on rural roads, many of the Massachusetts test sections were through developed areas. Several had curbs and gutters and high frequencies of intersections and driveways. Thus, results may not be directly comparable to those of other States.

Because of its larger data base, Alabama was subjected to an indepth investigation. All Alabama test sections had pavement widths that were either 22 ft (6.7 m) or 24 ft (7.3 m) with ADT between 2,000 and 5,000. Analysis showed that an 8-in (20-cm) edgeline on a 24-ft (7.3-m) pavement reduced edgeline-related accidents by approximately 10 percent when the pavement surface was wet. On the other hand, using the same 8-in (20-cm) edgeline on a 22-ft (6.7-m) pavement section there was a slight increase in edgeline-related accidents. This slight difference was probably due to sampling variations. Neither the decrease nor the increase was statistically significant at the 10-percent level.

Driver Response to Edgelines

To evaluate the effect of edgeline widths on driver performance, four two-lane 10-ft (3.04-m) wide sites with horizontal curves were selected in Massachusetts. Two observers were positioned in a parked vehicle along the side of the road at the test site. One observer determined whether vehicles coming around the curve departed from their lane by encroaching the centerline or edgeline. The second observer noted if there was an oncoming vehicle. Data were collected for both day and night conditions as well as for both right and left turn curves. For the before period, all four sites were striped with 4-in (10-cm) edgelines; for the after period, one site was restriped with 4-in (10-cm) edgelines and the other three were restriped with 8-in (20-cm) edgelines. Table 5 gives the raw data.

Table 3.—Summary of edgeline-related accidents by State and volume: 2,000 - 5,000 vpd

State	Control group		Treatment group		Apparent	Statistical
	Before	After	Before	After	treatment	significant
	4-in	4-in	4-in	8-in	effect (%)	change α=.10
Alabama	398	394	304	314	4	no
Maine	148	153	138	133	–7	no
Massachusetts	143	66	117	40	–26	no
New Mexico	356	162	233	115	8	no
Ohio	1,146	468	947	416	8	no
South Dakota	61	32	84	22	–50	no
Texas	119	90	92	72	3	no

vpd = vehicles per day.

Table 4.—Summary of edgeline-related accidents by State and volume: 5,000 - 10,000 vpd

State	Control group		Treatment group		Apparent	Statistical
	Before	After	Before	After	treatment	significant
	4-in	4-in	4-in	8-in	effect (%)	change α=.10
Maine	43	54	24	45	49	no
Massachusetts	92	36	174	58	–15	no
Ohio	829	336	845	374	9	no
Texas	108	48	50	38	71	no

vpd = vehicles per day.

The processed and compiled data were structured into a multidimensional contingency table (table 6). Loglinear modeling techniques, based on the principles of minimum discrimination information statistics, were used to test the hypothesis that lane departure is a function of the explanatory variables, i.e., day versus night, right versus left turn curves, 4-in (10-cm) versus 8-in (20-cm) width, and the presence or absence of oncoming vehicles. (*10*) Lane departure was defined as when a vehicle's wheel(s) touches or crosses—no matter how many times—either the centerline or edgeline while the vehicle travels through curve.

For each of the cells in table 5, a predicted value was calculated that best fit the data. These predicted values form the basis for calculating the odds shown in table 6. An odd is the ratio of the predicted number of lane departures to the predicted number of no departures. For example, for the before treatment period, the odd is 1.41 that a vehicle traveling at night through a curve in the left direction and seeing an oncoming vehicle will depart from its lane somewhere in the curve: for every 100 vehicles that do not depart their lane, 141 will depart from their lane of travel. The odd for the after period is 0.58, a considerable reduction.

The relative odd (the last column in table 6) is the ratio of the after treatment odd to the before treatment odd, measuring the change in the proportion of lane departures between the two periods. A relative odd of 1 means that there was no change in the proportion of lane departures; a relative odd of less than 1 means that there was a reduction in lane departures. The smaller the relative odd, the greater the reduction. Note that the relative odds show a greater reduction in lane departures with the 8-in (20-cm) edgelines than with the 4-in (10-cm) edgelines.

Service Life

Edgeline service life evaluations were conducted at 36 sections, half restriped with 4-in (10-cm) edgelines and half with the 8-in (20-cm). Serviceability evaluation was based on three criteria: appearance, durability, and night visibility in accordance with revised Standard ST-1. (*11*)

Because of the small data base, it was not possible to conclude that the 8-in (20-cm) edgelines last longer than the 4-in (10-cm) edgelines on two-lane rural roads. The Massachusetts evaluation indicated that there was no difference in serviceability between the two edgeline widths. In fact, after one winter, *all* the edgelines had to be restriped because of their rapid deterioration due to the use of deicing materials and abrasives. Several of the Maine test sections also had to be restriped for these reasons.

There was no clear difference in the serviceability between the two edgeline widths. Alabama and Ohio provided service life data for the 6th and 12th months after implementation. State evaluations show that the

8-in (20-cm) edgelines had a slightly higher rating than the 4-in (10-cm) edgelines, although both were acceptable. Both edgeline widths could probably last up to 2 years or longer. South Dakota service life data showed that after 1 year, the rating for night visibility was very low. When the edgelines were reevaluated 6 months later, however, the night visibility rating was higher than the previous 6 months. A possible explanation is that as the paint wore down, more glass beads were exposed to the surface, improving night visibility. New Mexico and Texas did not provide any service-life data.

Cost Assessment

The cost of installing edgelines varies considerably by State, year of installation, and whether the work is contracted out or performed by the State. The average incremental cost of striping 8-in (20-cm) edgelines rather than 4-in (10-cm) edgelines is between $200 and $250 per mile ($124 to $155 per linear km) in 1986 dollars. Material costs account for almost all of the difference. Labor and traffic delay is negligible. Based on the available data and identical service lives, to be cost effective, the 8-in (20-cm) edgelines must reduce accident costs by an amount equivalent to the increased material cost.

The average unit cost per accident established by the Federal Highway Administration for a run-off-the-road accident is $45,000 (1986 dollars). (*12*) Using the American Association of State Highway and Transportation Officials (AASHTO) unit accident costs, the average run-off-the-road accident would be $20,000. (*13*) With AASHTO's figure of $20,000, a service life of 1 year, and 1 edgeline-related accident every 2 miles (3.2 km) per year, the 8-in (20-cm) edgelines need to reduce edgeline-related accidents by 2 percent to be

Table 5.—Driver performance summary

Site	Curve Direction	Edgeline treatment (inches)	Time	Presence of opposing vehicle	Before treatment, Departed from lane: Yes	Before treatment, Departed from lane: No	After treatment, Departed from lane: Yes	After treatment, Departed from lane: No
1	Left	4	Day	yes	73	92	62	113
				no	69	51	61	73
			Night	yes	68	37	29	76
				no	68	41	49	60
2	Right	8	Day	yes	78	39	50	67
				no	51	40	25	66
			Night	yes	83	31	40	84
				no	87	51	51	87
3	Right	8	Day	yes	124	95	75	161
				no	84	50	38	156
			Night	yes	104	46	18	141
				no	93	17	23	100
4	Left	8	Day	yes	112	46	60	112
				no	76	51	60	70
			Night	yes	65	27	41	61
				no	109	17	67	66

Number of vehicle observations

1 in = 2.54 cm

cost effective; for the FHWA's figure a 1-percent reduction is needed. The cost of striping both edges of a roadway with 8-in (20-cm) edgelines for 100 miles would amount to $20,000—this equals AASHTO's average edgeline-related accident cost. Thus, for the wider edgeline to be cost effective, it must reduce the average number of accidents to below 50 for every 100 miles. Since the average cost applied to edgeline-related accidents varies considerably between the AASHTO and the FHWA figures, cost-effectiveness results must reflect the criteria within each method of estimation.

Summary

In general, 8-in (20-cm) edgelines are not cost effective for installation on two-lane rural roads in those areas where:

- There is frequent heavy snowfall and the use of deicing materials and abrasives tends to deteriorate edgelines.
- Pavement widths are less than or equal to 22 ft (6.7 m).
- Roads have paved shoulders over 6 ft (1.8 m) wide.

Study findings, however, suggest that 8-in (20-cm) edgelines could be potentially cost effective in reducing run-off-the-road accidents on two-lane rural roads with pavement widths of at least 24 ft (7.3 m), unpaved shoulders, and an average ADT of 2,000 to 5,000.

Based on the limited amount of data available, 8-in (20-cm) edgelines may be appropriate as a safety improvement when applied at spot locations such as isolated horizontal curves and approaches to narrow bridges.

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Table 6.—Odds of leaving lane versus staying in lane

Site	Curve direction	Edgeline width (inches)	Time	Presence of opposing vehicle	Odds: Before treatment	Odds: After treatment	Relative odds
1	L	4	Day	Yes	.87	.58	.67
1	L	4	Day	No	1.25	.83	.67
1	L	4	Night	Yes	1.41	.58	.41
1	L	4	Night	No	2.02	.83	.41
2	R	8	Day	Yes	1.78	.61	.34
2	R	8	Day	No	1.28	.44	.34
2	R	8	Night	Yes	2.88	.61	.21
2	R	8	Night	No	2.07	.44	.21
3	R	8	Day	Yes	1.38	.44	.32
3	R	8	Day	No	1.54	.23	.15
3	R	8	Night	Yes	2.07	.16	.08
3	R	8	Night	No	6.31	.23	.04
4	L	8	Day	Yes	1.70	.58	.34
4	L	8	Day	No	2.43	.83	.34
4	L	8	Night	Yes	2.74	.83	.21
4	L	8	Night	No	3.94	.83	.21

1 in = 2.54 cm
1 mi = 1.61 km

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Model Study of the Hatchie River U.S. 51 Bridge

J. Sterling Jones, P.E.

Introduction

Spans of the northbound U.S. 51 bridge over the Hatchie River collapsed on April 1, 1989. Five vehicles went into the river, and eight people were killed as a result of the collapse.

To help determine the cause of the bridge's collapse, the Federal Highway Administration (FHWA) was asked by the National Transportation Safety Board (NTSB) to conduct hydraulic model studies of bent 70 of the U.S. 51 bridge. (*1*)[1] Only bent 70 was tested because onsite investigation established that the failure of this bent probably triggered the collapse.

Bent 70 was a two-column bent with independent footings. Each footing was 6.5 ft (1.98 m) square and had five untreated timber piles. The piles were assumed to be 1 ft (.30 m) in diameter and 20 ft (6.1 m) in length. Bent 70 was designed with a floodplain foundation that was not intended to be in the main channel. Over the years, however, the channel migrated laterally, putting bent 70 in the main channel and subjecting it to local scour and the effects of debris as well as a basic loss of foundation support.

A 1:20 scale model of bent 70 was tested during the summer of 1989 in the FHWA hydraulics laboratory. The study had two basic objectives:

- To determine how the maximum local pier scour that may have occurred after the channel migrated to bent 70.
- To obtain videotape shots of the local scour process for use as a visual aid in the NTSB public hearing conducted to gather evidence concerning the collapse.

[1] Italic numbers in parentheses identify references on page 165.

Description of Tests

The bent was tested three ways to get relative depths of scour under various conditions. In the first test, the footing was placed above the stream bed to simulate approximately 5.9 ft (1.8 m) of exposed pile (figure 1). The bottom of the footing for bent 70 was determined to be at elevation 237.9 ft (72.5 m) from the bridge plans. The exposed pile length was determined by assuming that the lowest channel bottom elevation (232.0 ft [70.7 m]) could have occurred at bent 70 prior to the collapse.

The second test maintained the position of the footing but added debris around the upstream footing. The selection of debris was rather arbitrary since there were no photographs of the actual pier debris and divers were unable to get to the piles after the collapse (figure 2). The debris used in the model study was chosen to simulate the type of debris that had been sketched during one of the bridge inspections.

The third test was strictly a bench mark experiment with the square pier extending into the bed with no exposed footing or piles—i.e., the condition that would have prevailed if the channel had not migrated. Prediction equations exist to compute scour for a regular square pier, but such equations do not exist for an irregular shape with an exposed footing, exposed piles, and a pier; all obstructing the flow. This bench mark configuration was used to determine if the exposed footing and piles resulted in more or less scour than the regular square pier.

Figure 1.—Laboratory set-up for the first test with exposed piles.

Figure 2.—Model with simulated debris around the upstream piles used in the second test.

Scaling Ratios

A 1:20 scale model was selected primarily on the basis of the depth of flow that could be accommodated in the Turner-Fairbank Highway Research Center (TFHRC) flume: the depth of flow in the river at the time of failure was around 22 ft (6.7 m); since no more than 18 in (45.72 cm) of water could be held in the flume, a 1:20 model ratio was a convenient choice.

The Froude model law is assumed to hold for free surface flow where gravitational forces dominate over viscous and surface tension forces. (*2*) Under this law, the Froude number, $V/(gy)^{0.5}$, correlates model and prototype measurements where V is velocity, g is the acceleration of gravity (this is normally the same for both the model and the prototype), and y is the depth of flow or other characteristic length dimensions. Accordingly, the following ratios are used to relate model and prototype measurements:

$$\frac{\text{(depth of flow) model}}{\text{(depth of flow) prototype}} = L_r = 1/20$$

$$\frac{\text{(bent dimensions) model}}{\text{(bent dimensions) prototype}} = L_r = 1/20$$

$$\frac{\text{(velocity) model}}{\text{(velocity) prototype}} = (L_r)^{0.5} = 1/4.47$$

$$\frac{\text{(time) model}}{\text{(time) prototype}} = (L_r)^{0.5} = 1/4.47$$

$$\frac{\text{(depth of scour) model}}{\text{(depth of scour) prototype}} = L_r = 1/20$$

Experimental Procedure

The tests were run in a tilting flume located in the FHWA hydraulics laboratory at the TFHRC. The flume is 6 ft (1.8 m) wide, 70 ft (21.3 m) long, and 2 ft (0.6 m) deep. It is equipped with a depressed section near the middle of its length for a sand bed for scour experiments. Because the flume has a built-up false floor, its full depth is not available for water flow; only about 18 in (45.7 cm) of its 2 ft (0.6 m) can be used. The flume has a recirculating system for the water but not for the sediment.

Since the sediment could not be recirculated in the flume and the bed material from the site could not be modeled at this small scale, the experiment was designed to observe relative maximum pier scour measurements. Accordingly, velocities were selected to course incipient motion of the bed material used in the experiment. Local pier scour is nearly maximum at the velocity that causes incipient motion on the bed material. Higher velocities bring in bed material from upstream to replenish additional pier scour.

The limitation of these types of experiments is that the velocity cannot be varied. The velocity used in the model study is determined by the bed material rather than the actual velocity that occurred at the time of failure. Thus, the resulting scour that was measured in the model study represents a maximum local pier scour that *may* have occurred; it does not necessarily represent the *actual* pier scour at the time of failure. Although the scour measurements may not represent actual scour, the relative scour measurements for the irregular exposed footing and the bench mark regular square pier are useful in estimating what occurred at the site.

The incipient motion velocity was obtained for each test by a gradual process of first filling the flume to the desired depth and then adjusting discharge and tailgate settings. A fire hose was used to backfill from the downstream end at approximately the same rate as the pump that filled from the upstream end. This procedure was used to avoid scouring a big hole around the pier with a high velocity surface wave during the filling process. After the flume was filled to the 1-ft (0.3-m) depth, the pump discharge rate was increased as the tailgate was lowered to increase the velocity until the bed material just started to move.

The experiment was then run with a steady discharge for 2 hours; this is equivalent to 8.94 hours in the prototype. After the test, the flume was drained slowly and the scour measurements were made.

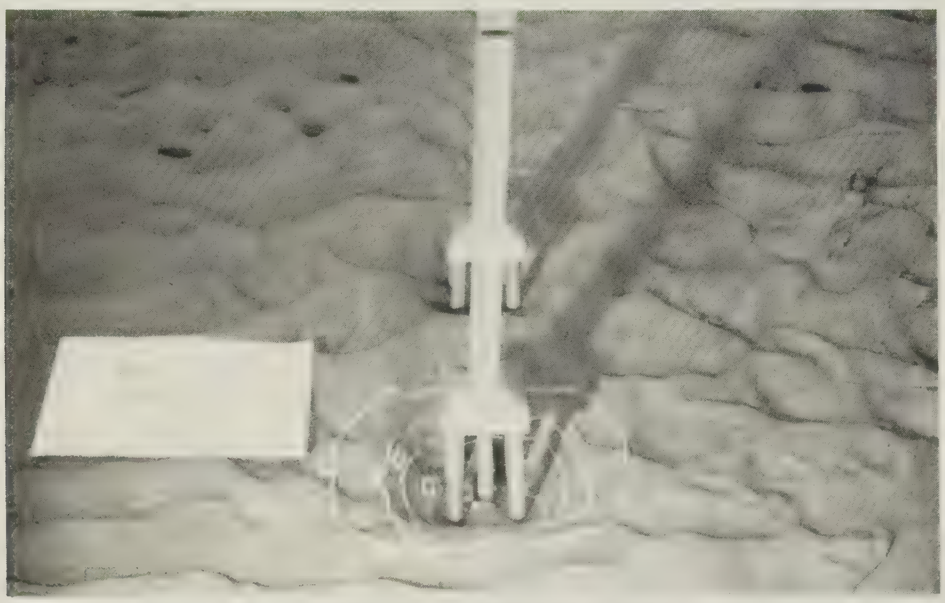

Figure 3.—Contour intervals that are equivalent to 1-ft (.30 m) intervals for the prototype after the test with debris on the upstream piles.

The scour holes were contoured by placing yarn around the water edge as the level dropped to predetermined gauge readings to match 1-ft (0.30-m) intervals for the prototype (0.05-ft [0.015-m] intervals for the model). Figure 3 illustrates the contours that were marked after the second test that was run with debris on the upstream pier. White yarn was used to illustrate scour contours; black yarn was used for deposition which occurred downstream of each pier.

The model debris was a bundle of sticks that were slightly longer than the width of the footing. The sticks were soaked in water for 3 days so they would tend to sink rather than float during the test. The debris was tied with wire to the piles near the original bed elevation.

Dye Experiments

Red food coloring was mixed with water and injected along the upstream face of the pier to trace the diving currents that are normally associated with pier scour. When the dye was injected along the face of the pier *above* the footing, it dived to the top of the footing but dissipated into the flow before it reached the bed. Only when it was injected *below* the top of the footing did it dive to the bed level at the piles. Since the footing extended out from the pier by one pier width in each direction, it effectively dissipated the dividing current from the pier. A second flow pattern was set up in the lower flow zone by the footing itself and the underlying piles.

Although no experiments were conducted to this end, the scour holes that formed around the exposed piles probably would have been the same with or without the piers above the footings. Also, the footing would probably have been a highly effective scour suppressor had it been at below-the-bed elevation.

Results

The maximum depth of local pier scour for the exposed piles without debris was 0.164 ft (0.050 m) in the model; this is equivalent to 3.3 ft (1.0 m) for the prototype. The maximum point of scour was the middle pile of the upstream foundation which was directly below the center of the footing. Scour at the downstream foundation was about 40 percent less than at the upstream foundation. A mound of material was deposited under the bent between the piers in all three of the tests as illustrated in figure 4. The maximum scour around the square pier that extended into the stream bed was 0.214 ft (0.065 m), which is equivalent to 4.3 ft (1.3 m) for the prototype. The pier scour equation from reference (*3*) is:

$$\frac{y_s}{y_1} = 2.0\ k_1\ k_2 \left(\frac{a}{y_1}\right)^{0.65} \left(\frac{v_1}{\sqrt{g y_1}}\right)^{0.43} \quad (1)$$

where:
- y_s = depth of scour
- y_1 = depth of flow = 1.0 ft (.30 m)
- k_1 = 1.1 for a rectangular pier
- k_2 = 1.0 for aligned flow
- a = width of piers = 0.10 ft (.03 m)
- v_1 = velocity approaching the pier = 1.24 ft/sec (.378 m/sec)

This equation would predict the depth of scour to be 0.256 ft (0.078 m) for the square pier model which is equivalent to 5.1 ft (1.6 m) for the prototype. That is reasonably good agreement for a small-scale model. There is no corresponding prediction equation for the complex pier footing and exposed pile combination that existed at the Hatchie River. These experiments do indicate, however, that the local scour for that situation was somewhat less than it would have been for the pier itself.

The debris made surprisingly little difference in the maximum scour depth in the second test. The maximum scour around the center pile was 0.168 ft (0.051 m) with debris, compared to 0.164 ft (0.050 m) without debris—this is not a significant difference for these types of experiments. As placed, the debris tended to shield the center pier where the maximum scour occurred and partially offset the extra obstruction to the flow. Had the model debris been a denser clump, it would have increased scour considerably. The results of the laboratory experiments are summarized in tables 1 and 2 and illustrated in figures 5, 6, 7, and 8. The lateral extent of scour was measured from the corners of the footing in each direction for the first two tests for the exposed footing. It was measured from the center of the four faces of the square pier for the third test with the pier only extended.

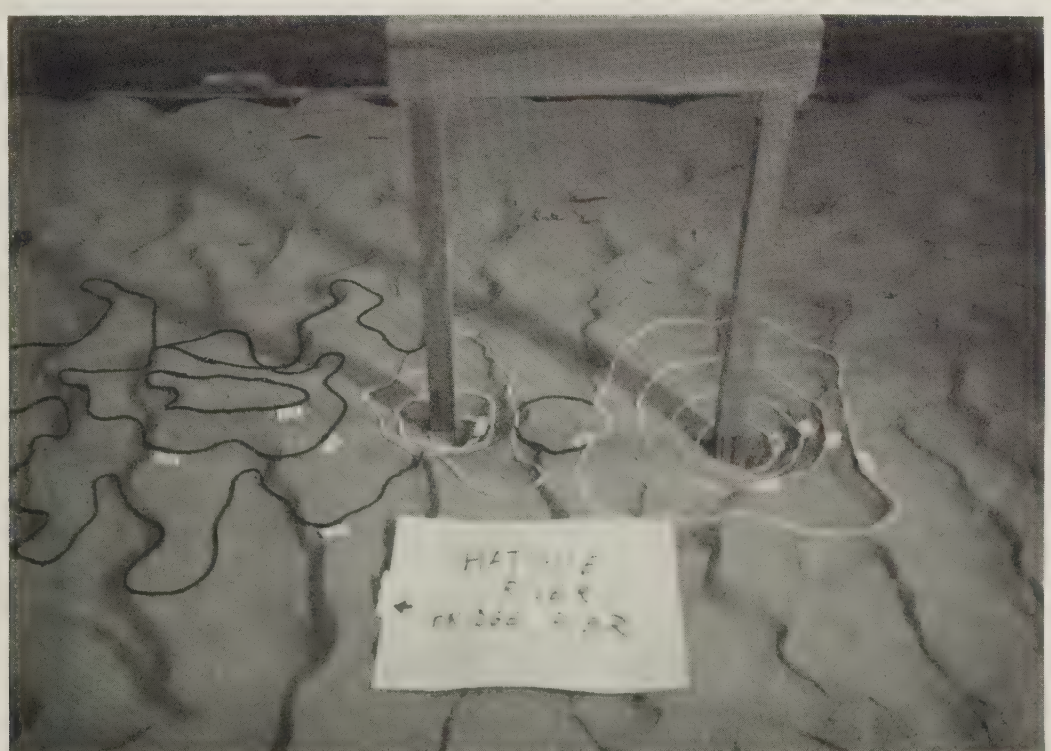

Figure 4.—Scour around piers extended into the bed material (dark contours represent deposition).

Conclusions and Recommendations

• Although local pier scour was not the primary element in this bridge collapse, it probably was a contributing factor for a foundation that was already vulnerable once the channel migrated laterally.

• Local pier scour from the complex foundation was no more than it was from the simpler and smaller square column by itself. If a portion of a foundation extends out as a shelf in the lower flow zones, it can act as a scour arrester to counteract the extra obstruction to the flow.

Note:
pier width
extension

$$\frac{\frac{6.5\text{ ft}}{2}}{2\text{ ft}} = 1.6\text{ pier widths} \qquad (2)$$

1 ft = .30 m

• Although not related to this model study, one recommendation for bridges such as this one which have some channel migration and unknown local scour during floods is to develop convenient instrumentation to facilitate recording stream bed cross sections just upstream and downstream of the bridge from the bridge deck. Computer plotting routines should be developed to superimpose these cross sections quickly with foundation elevations and to compare the cross sections from different inspections of the bridge.

Table 1.—Summary of scour depth measurements for 1:20 model

Exposed piles
(depth = 1.0 ft, approach velocity = 1.26 ft/sec)

Column	Pile	Scour depth (ft)
Upstream	Upstream	0.15
	Middle	0.164
	Downstream	0.144
Downstream	Upstream	0.06
	Middle	0.096
	Downstream	0.042

Exposed piles with debris around upstream footing
(depth = 1.0 ft, approach velocity = 1.279 ft/sec)

Column	Pile	Scour depth (ft)
Upstream	Upstream	0.116
	Middle	0.168
	Downstream	0.066
Downstream	Upstream	0.06
	Middle	0.10
	Downstream	0.044

Piers only—bench mark test
(depth = 1.0 ft, approach velocity = 1.242 ft/sec)

Column	Scour depth (ft)
Upstream	0.214
Downstream	0.144

1 ft = .3048 m
1 ft/sec = .3048 m/sec

Table 2.—Summary of scour extent from edge of footing

Exposed piles
(depth = 1.0 ft, approach velocity = 1.26 ft/sec)

Footing	Upstream side		Right side		Left side		Downstream side	
	Left	Right	US	DS	US	DS	Left	Right
Upstream	.39	.38	.373	.437	.37	.436	.36	.40
Downstream	.05	.06	.15	.165	.12	.087	.14	.12

Exposed piles with debris around upstream footing
(depth = 1.0 ft, approach velocity = 1.279 ft/sec)

Footing	Upstream side		Right side		Left side		Downstream side	
	Left	Right	US	DS	US	DS	Left	Right
Upstream	.32	.33	.35	.35	.27	.28	.13	.13
Downstream	.14	.15	.173	.188	.185	.145	.17	.17

Piers only—bench mark test
(depth = 1.0 ft, approach velocity = 1.242 ft/sec)

Footing	Upstream side	Right side	Left side	Downstream side
Upstream	.38	.523	.578	.41
Downstream	.21	.188	.261	.20

1 ft = .3048 m
1 ft/sec = .3048 m/sec

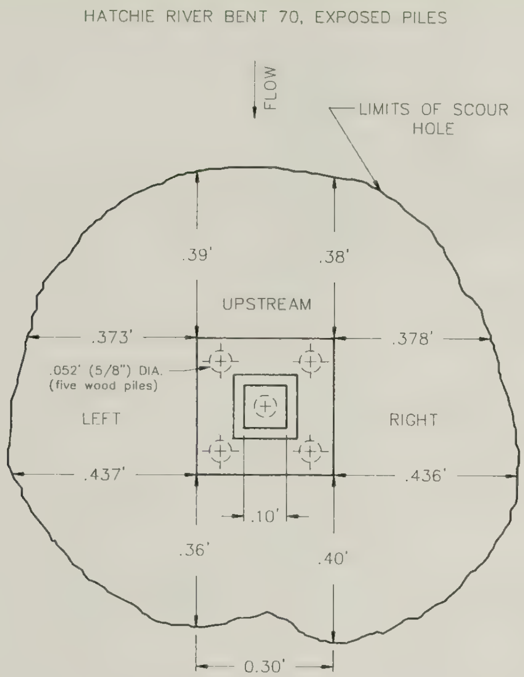

Figure 5.—Plan view of scour hole, upstream footing, no debris.

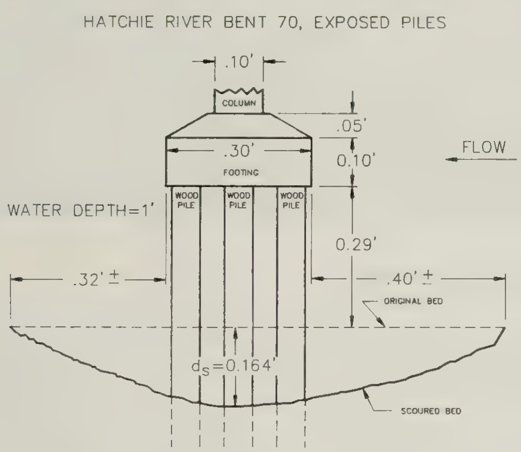

Figure 6.—Profile of scour hole, upstream footing, no debris.

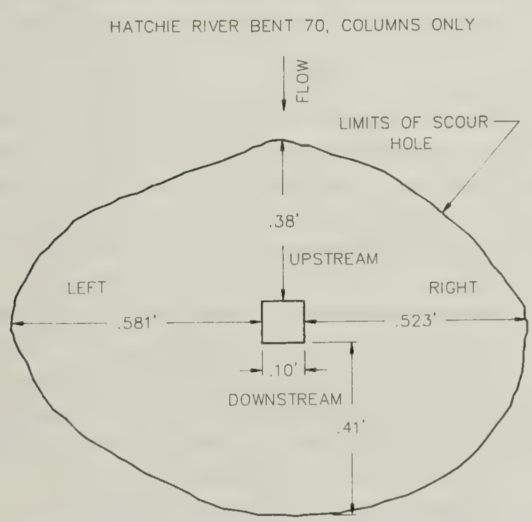

Figure 7.—Plan view of scour hole, upstream column.

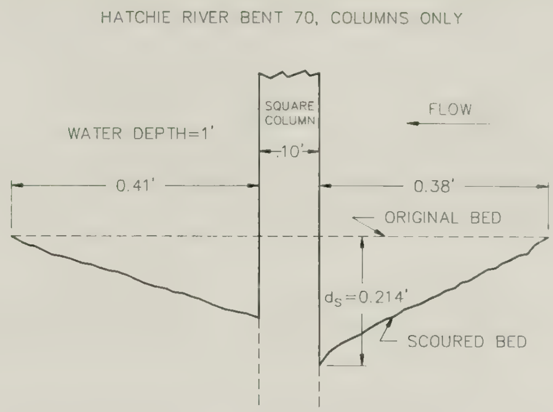

Figure 8.—Profile of scour hole, upstream column.

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(*1*) Philip L. Thompson. "April 1989 Hatchie River U.S. 51 Bridge Failure," unpublished report submitted to the National Safety Board, Washington, DC, September 1989.

(*2*) "Hydraulic Models," *ASCE Manual of Engineering Practice, No. 25* (reprint), American Society of Civil Engineers, New York, 1963.

(*3*) "Scour at Bridges," *Hydraulic Engineering Circular, No. 18* (unpublished draft), Federal Highway Administration, Office of Engineering, Bridge Division, Washington, DC, November 1989.

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Field Evaluations of Breakaway Utility Poles

by Janet A. Coleman[1]

Introduction

Each year approximately 1,500 people are killed as a result of collisions with utility poles making utility poles second only to trees and shrubs as the most frequently struck roadside objects involved in fatal accidents (table 1). It has been estimated that 88 million utility poles are located in the right-of-way of roads and streets. As part of its overall highway safety improvement program, the Federal Highway Administration (FHWA) is investigating methods of reducing hazards posed by utility poles.

One common way to reduce the hazard of a manufactured roadside obstacle is to make the object yield or break away when struck by a vehicle. This breakaway concept was first developed in the 1960's for large freeway sign supports. The sign post is designed to give or break away upon impact and allow the vehicle to drive safely underneath the sign. The impact with the sign post slows the vehicle down slightly, allowing it to come to a safe stop. This concept is also used in highway luminaire supports.

The success of the breakaway concept for sign and luminaire supports led to investigations to see if it could also be used with timber utility poles. Because utility poles may support power lines, they pose a slight complication. Pole design modifications must keep power lines from falling (thereby helping to minimize service interruptions); maintain the pole's capacity to withstand environmental loads such as snow, ice, and wind; and allow a vehicle's occupants to survive an impact.

[1]This article is adapted from a presentation made to the International Right-of-Way Association's Educational Seminar in Dearborn, Michigan, on June 19, 1989.

Table 1.—Distribution of fatalities in fixed object collisions in 1988

Fixed object	Fatalities	Percentage of fixed object fatalities
Tree or shrub	3,328	25.32
Utility pole	1,476	11.23
Guardrail	1,384	10.53
Embankment	1,360	10.35
Curb or wall	892	6.78
Ditch	825	6.28
Culvert	650	4.95
Sign or luminaire support	576	4.38
Bridge or overpass	553	4.21
Miscellaneous pole or support	501	3.81
Fence	482	3.67
Other fixed object	1,116	8.49
Totals	13,143	100.00

Over the past 15 years, several efforts have been made to adapt the basic breakaway concept to the unique requirements of utility poles. Designs such as holes, grooves, and saw cuts were tried to make the utility pole weaker at the base so it would give way when hit by a vehicle. The slip-base concept was demonstrated using both pendulum and vehicle impacts and was successful.

The mid-portion of the pole, below the wires, needed to be modified so it would also give or bend and allow the utility lines to remain functional. A number of different hinge designs and locations were tried to allow the pole to bend after a vehicle stuck it.

Field Testing

Following crash testing, it was concluded that the slip-base design was the most suitable in terms of reducing the severity of vehicle impact and withstanding environmental loads. Additional work was done that included full-scale crash testing on poles that carried joint electric and telephone lines. During research completed in 1984 by the Texas Transportation Institute, multiple crash tests were conducted on modified poles that included slip-base lower connections, upper hinge connections, and support cables. Support cables help stabilize the upper part of the pole during vehicle impact and facilitate activation of the upper hinge. The idea of support cables was based on work done in Australia. Both single and multiple support cables were tested.

Following the research in Texas, which successfully tested the breakaway design for timber utility poles, the FHWA Office of Implementation contacted States to determine interest in performing a multiyear field evaluation of the breakaway utility poles. Kentucky and Massachusetts expressed interest and agreed to evaluate the breakaway poles for a 2-year period.

To date, the Kentucky Utilities Company, working in conjunction with the Kentucky Transportation Cabinet and the University of Kentucky, has retrofitted 10 poles in the Lexington area with the breakaway hardware. Together the university staff and the utility company will monitor pole performance and prepare the required final reports and documentation. Most of the Kentucky poles have been in place for almost 2 years—with no hits reported. In Kentucky, the university staff periodically visits the pole locations and checks for evidence of unreported accidents. So far there have been none.

Massachusetts has selected 20 sites and in late October 1989 installed the first 4 new poles with the slip-base hardware in the Newbury area on the north shore. Existing poles were replaced with new poles that included the breakaway hardware.

As part of the Massachusetts work, the State and the utility companies have done additional load testing of the poles and also made some modifications to the design of the upper and lower connections. This will provide additional input on the ability of the present breakaway design to meet safety and utility operational needs.

Installation Process

Initially, Kentucky Utilities planned to replace existing poles with new ones with the breakaway hardware. After reviewing the work entailed by replacement, the company instead decided to retrofit existing poles. The retrofit work consisted of the installation of the three components of the breakaway system:

- Slip-base connection.
- Upper hinge connection.
- Upper support (guy) wires.

The utility company's five-person crew needed about 6 hours to install the slip-base and upper hinge connections. The time to install the guy wires varied depending upon the number of tree limbs with which the crew had to contend. This work generally took a few hours. The installation was completed in the following manner:

- Various traffic control devices were used at the work zone site. The curb lane was closed to traffic by the use of flashing arrow boards, traffic cones, and an advance warning sign. The utility company trucks and equipment were parked in the curb lane. For this installation, a digger derrick truck and a double bucket truck were required.

- As shown in figure 1, the upper part of the utility pole was stabilized by the boom arm of the digger derrick truck while work was performed on the bottom part of the pole. The crew then removed about 30 in (76.2 cm) of soil from around the pole's base. This gave the crew room to make a cut through the pole and ensured adequate space to put the lower half of the slip-base connection over the pole stub in the ground.

- A chain saw was used to cut through the utility pole about 3 in (7.62 cm) above ground (figure 2). This elevation is intended to minimize the chance that the bottom of a vehicle will snag when going over the stub following a collision.

After cutting through the pole, notches were cut into the stub to provide more room for the bonding material. The bottom half of the steel slip-base connection then was placed over the stub.

- The two-component bonding material was mixed at the site as shown in figure 3. It was combined and used quickly because it sets within a few minutes. The bonding mixture was poured through the hole in the center of the slip base. It expanded very quickly to fill the space between the timber stub and the slip-base connection.

- Next, the upper half of the slip-base connection was installed on the above-ground portion of the pole. To do this, the above-ground portion was held out to the side while the crew placed the slip base over the pole (figure 4). A very thin [1/64th of an inch (.0397 cm)] keeper plate was then placed between the upper and

Figure 1.—Work zone site.

Figure 2.—Crew members cutting through the utility pole.

Figure 3.—Bonding material mixed at the site.

Figure 4.—Installation of the upper half of the slip-base connection.

lower portions of the slip-base connection. The keeper plate is designed to prevent the top part of the slip-base connection from creeping off the lower portion if the bolts become loose over time. The upper and lower parts of the slip-base connection were then bolted together using six 1 1/8-in (2.86-cm) diameter bolts.

Adjustment bolts were used to center the upper portion of the pole. These bolts were removed at the end of the job. Additional bonding material was mixed and used to set the upper part of the slip-base connection. The crew used tape around the top of the steel slip-base connection to help in cleaning up any spills. The excess amounts of bonding material were trimmed.

• As shown in figure 5, the upper hinge connection was installed. Four 1-in (2.54-cm) diameter holes were drilled through the pole where the steel collars were to be placed. Then, one of the crew members cut through the pole using a chain saw. The crew had partially assembled the upper hinge connection on the ground. They placed the partially assembled connection over the two ends of the cut pole and bolted it in place. These bolts were tightened with a torque wrench to 100 ft·lb (34174 ft·pdl).

• Steel support cables were run from the retrofitted pole to adjacent poles (figure 6). One cable was fastened above the upper hinge mechanism; and the other was fastened below the cross arms. The support wires were fastened to the poles with a thimble eye bolt and nut and preformed guy grips.

• To complete the job, the slip-base bolts were tightened with a torque wrench to 200 ft·lb (68348 ft·pdl). The crew then regraded the soil around the base of the pole and removed the traffic control devices.

Figure 5.—Installation of the upper hinge connection.

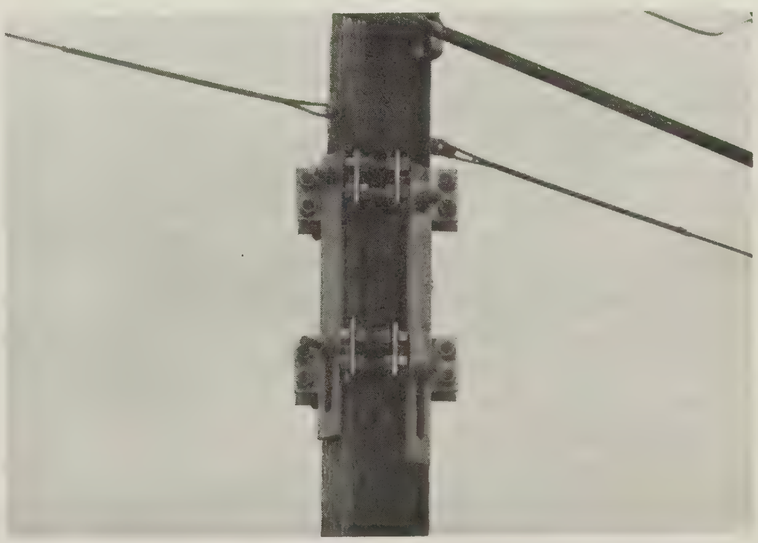


Figure 6.—Steel support cables.

Information and Data Collection

When all the poles have been installed, the utility companies and the States will need to gather certain data to monitor pole performance. Both the States and the utility companies are collecting the following types of data:

- *Preparation.* Provide information on how the sites were selected. What criteria were used to select or rule out sites? What were the characteristics of the sites selected?

- *Installation.* Describe the installation process used. Were existing poles retrofitted or replaced? Why? How long did the installation take? What size crew and what types of equipment were used? What special equipment or tools were needed? What problems were encountered and how were they overcome? What changes were made, if any, in the installation procedures over time? Finally, what changes would States and utility companies make in future installations (including any changes in hardware)?

- *Monitoring.* Describe the events of the evaluation period, particularly any reported (or unreported) accidents or vandalism. The evaluation may need to be extended to provide accident information.

- *Documentation.* Provide videotapes, films, or slides as well as the reports prepared by the States to document the installation procedures and maintenance practices. In the event of an accident involving a breakaway pole, the States may want to prepare a video or film to document the extent of damage to the pole and—if possible—to the vehicle. Accident severity in terms of fatalities and injuries should be documented, as should the costs of repairing the pole and the vehicle.

- *Utility company requirements.* The utility company needs to check the tightness of the bolts on the upper and lower connections and monitor any operational problems experienced. Only the utility company can report the degree of difficulty involved in installing and maintaining the slip-base utility poles. Companies should therefore provide information as to whether the installation involved more work than they anticipated. In the event of an accident, how soon were companies able to repair the pole? Do they plan to immediately replace the breakaway pole hardware or install a standard pole until a special crew can repair or replace the breakaway hardware? How many crews are companies going to train? What standards have they established for pole maintenance?

- *Environment.* Other useful documentation would include records identifying the days on which power lines were exposed to heavy ice or snow loads, strong winds, or rain. Any installations possibly damaged due to severe weather should be investigated and documented.

Conclusion

It is too soon to tell what conclusions will come from these evaluations, but it is expected that:

- Vehicle occupants involved in crashes with breakaway utility poles will likely survive the impact.

- The poles will successfully perform in multidirectional hits.

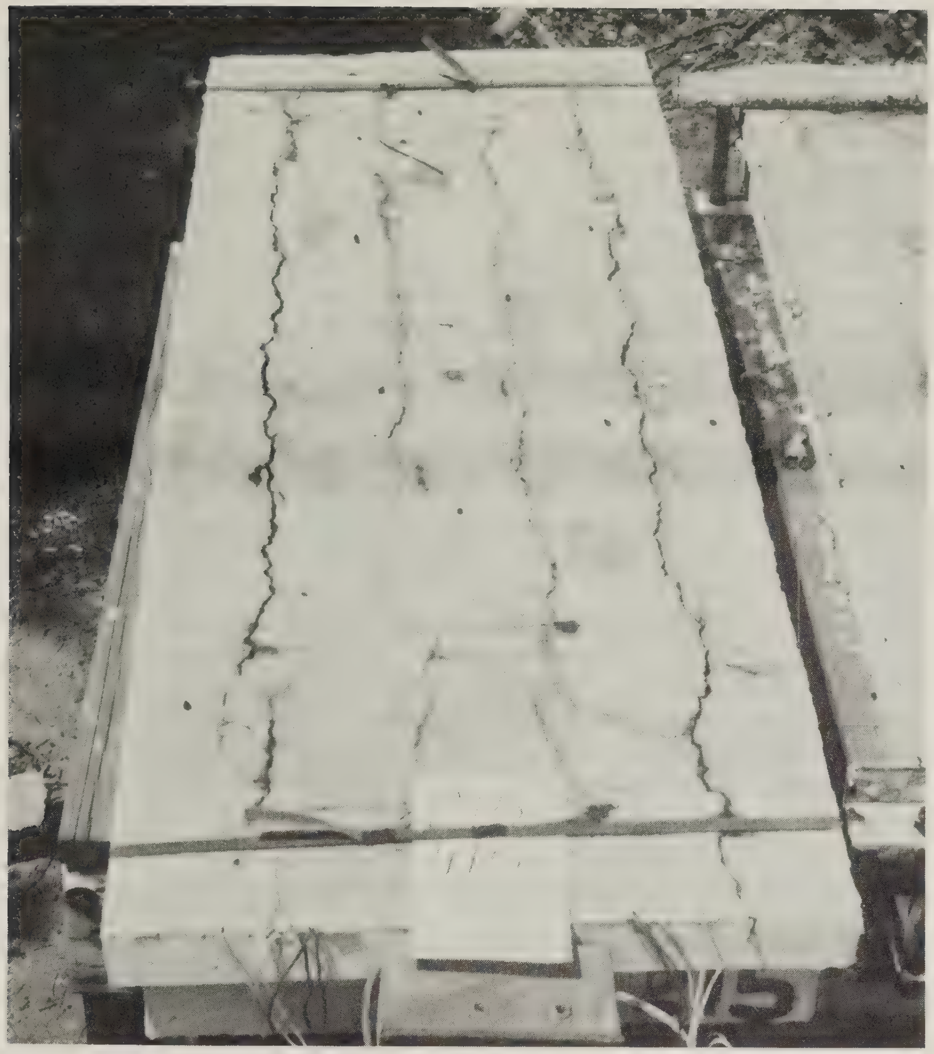
- The poles will adequately withstand environmental loads.

- Operational problems encountered by utility companies will be overcome.

- The breakaway utility pole will be an acceptable countermeasure to help reduce utility pole accident severity.

The final reports from Kentucky and Massachusetts are scheduled to be available in late 1992. The States may choose to extend their evaluation periods if there have not been any reported accidents at the sites. Even if there are no reported accidents by the end of the originally scheduled 2-year evaluation period, both States and the utility companies should have a good idea of the difficulties involved in installing and maintaining the breakaway poles and any operational problems involved. The 2-year evaluation period should be adequate to see how the poles perform under various wind, ice, and snow loads.

Janet A. Coleman is a program analyst in the Office of Implementation of the Federal Highway Administration. She is currently involved in implementing research results in the following safety areas: trucks, roadside hardware and design, railroad-highway grade crossings, utility poles, and pedestrians. She has a B.S. and an M.A. in Mathematics from Boston College.



Effectiveness of Calcium Nitrite Admixture as a Corrosion Inhibitor

by Yash P. Virmani

Introduction

The life of bridge decks and substructures is significantly reduced by the corrosion of unprotected (black) steel. Corrosion inhibition can be achieved after the mechanism of corrosion and the factors affecting it are understood. Recent research has helped explain the corrosion phenomenon for black steel rebars in salt-contaminated concrete when oxygen and water are present in optimum quantities. (*1*)[1]

It is generally accepted that the principal cause of corrosion is the occurrence of macroscopic corrosion cells between the top and bottom mats of rebars (those in which large quantities of steel drive corrosion at the anodic steel). Microscopic cells that occur locally (i.e., those that operate on a small section of the same reinforcing rebar) are less important. The presence of chloride ions in the concrete produces an electrical potential difference at the top mat (chloride-contaminated concrete) and between the top and bottom mats (chloride-free concrete). This difference produces the flow of an electrical (corrosion) current. The amount of iron consumed in forming the various corrosion products is directly related to this measured corrosion current. The corrosion rate is controlled by the ability of the cathodic bottom mat to reduce oxygen while the anodic top mat is subject to iron loss.

The addition of calcium nitrite has been shown to improve significantly the long-term corrosion resistance of black steel in chloride-contaminated concrete. (*2*) Although calcium nitrite has not been shown to delay the *initiation* of corrosion significantly, it does substantially reduce the *severity* of subsequent corrosion in black steel and strands. (*3*) Moreover, calcium nitrite, when compared with calcium chloride as an accelerator for curing of concrete mixtures, significantly delayed the onset of corrosion of black steel reinforcement and subsequent concrete cracking. (*4*) A Louisiana Department of Transportation study concluded that, as a corrosion-inhibiting admixture, calcium nitrite is beneficial in protecting black steel. (*5*)

In 1980, the Federal Highway Administration (FHWA) initiated an outdoor research study using calcium nitrite as an admixture in salty concrete to inhibit the corrosion of black steel reinforcing rebars. (*6*) This research was motivated by the need for an alternative corrosion protection system

[1] Italic numbers in parentheses identify references on page 182.

to mitigate the severe corrosion of black reinforcing steel rebars in marine and deicing salt environments. This article presents the long-term results from the use of this admixture to inhibit the corrosion of black steel reinforcing rebars in chloride-contaminated concrete. In a comparison series of test slabs, epoxy-coated rebars were used as another alternative corrosion protection system. The long-term results on that system will be reported in the future.

Description of Specimens

Eighteen slabs were used in this test series. Five slabs with no calcium nitrite were control specimens; these were designated as numbers 211 through 215. The remainder were test specimens designated as numbers 216 through 228 and had 2.75 percent of calcium nitrite by weight of cement in both the top and bottom lifts of concrete. Nominal chloride contents of 0, 5, 10, 15, 20, 25, and 35 lb Cl^-/yd^3 (0, 3, 6, 9, 12, 15, and 21 kg Cl^-/m^3) were mixed in the top lift concrete of the calcium nitrite slabs. The control slabs were fabricated with 0, 5, 15, and 35 lb Cl^-/yd^3 (0, 3, 9, and 21 kg Cl^-/m^3).

The research design posed a severe corrosion environment: chloride was added directly to the fresh concrete thus complicating the reaction mechanism for normal passivation of the steel by concrete and the nitrite ions. In addition, the concrete quality (water to cement ratio of 0.53) was quite poor.

Each slab was 2 ft by 5 ft by 6 in (60 by 150 by 15 cm) thick and cast in two lifts. The lower lift was 3.5 in (8.75 cm) thick and cast with chloride-free concrete. The 2.5-in (6.25-cm) thick top lift contained varying amounts of admixed chloride ions and was cast 1 to 3 days after the lower lift. The upper lift was reinforced by a mat of rebars, consisting of four number 5 longitudinal bars 51 in (127.5 cm) long and two number 4 cross bars 18 in (45 cm) long. The lower mat consisted of seven number 5 longitudinal bars 51 in (127.5 cm) long and three number 4 cross bars 18 in (45 cm) long. The slab design is shown in figure 1. All rebars met American Association of State Highway and Transportation Officials (AASHTO) specification M-3l. The clear concrete cover over the top mat was 3/4 in (1.88 cm) with a 1-in (2.5-cm) concrete cover below the bottom mat, leaving a clear 2 in (5 cm) of concrete between the two mats.

The concrete in each slab had a mixing ratio of 1:1.76:2.36 for cement, fine and coarse aggregates, by weight. Detailed properties of the concrete mix design are shown in table 1. The fine aggregate was white marsh sand which has a specific gravity of 2.64 and a fineness modulus of 2.6. The coarse aggregate was riverton limestone which has a specific gravity of 2.77 and a 3/4-in (1.88-cm) maximum size, graded to the midpoint of the AASHTO M-43 size number 67 specification.

All coarse aggregates were separated into four sizes which were then batched separately to ensure gradation control. The concrete was mixed in 9-ft^3 (0.295-m^3) batches in an 11-ft^3 (0.36-m^3) rotary drum mixer and placed in two lifts. The lower lift was cured with wet burlap and then wire brushed prior to the placement of the top lift. The top lift was also cured with wet burlap and polyethylene for 14 days. The slabs were mounted on 3-ft (0.91-m) posts at the FHWA outdoor exposure site.

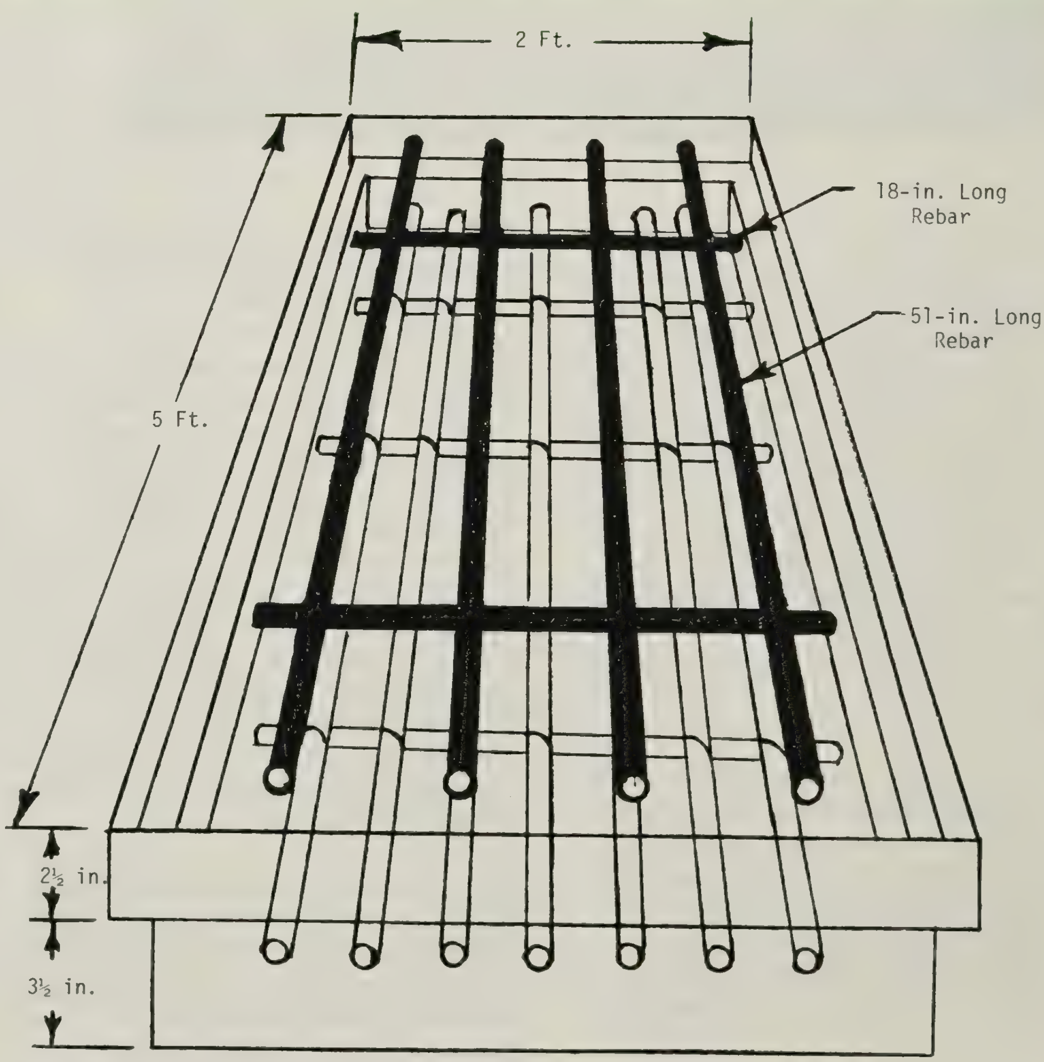


Figure 1.—Standard slab design.

Table 1.—Concrete mixture design[a,b]

Cement	7.0 sacks/yd^3
Water to cement ratio	0.53
Fine aggregate	1,160 lb/yd^3
Coarse aggregate	1,550 lb/yd^3
Darex AEA	290 ml/yd^3
Unit weight	138 lb/ft^3
Air content	7±1.5 percent
Slump	2.0 to 3.0 in

[a]Made in 9-ft^3 batches.
[b]Calcium nitrite slabs contain 18.09 lb of calcium nitrtie dissolved in 43.2 of water. This amount is subtracted from the total water added in the concrete mixture.

1 in	=	25.4 mm
1 ft^3		0.028 m^3
1 sack/yd^3	=	94 lb/yd^3 = 56.4 kg/m^3
1 lb/yd^3	=	0.6 kg/m^3
1 lb/ft^3	=	16.02 kg/m^3

Instrumentation

Number 12 gauge stranded copper lead wires with teflon insulation were used to connect the rebar mats to the switch box. To attach the bars their ends were sand blasted, a 1-in (2.5-cm) wide area on one side of each bar was flattened, and a 1/4-in (0.63-cm) diameter hole was drilled through the bar to receive a 1/4-in (0.63-cm) bolt for fastening the lead wire. The attachment area was then well coated with epoxy. Electric leads were attached to all top mat rebars and to two of the bottom mat rebars before the concrete was cast.

To facilitate corrosion measurement, all lead wires were extended outside the concrete. The switch was kept at the "on" position to maintain the electric coupling at all times except when gathering data. Nine thermocouples were embedded in each slab for temperature monitoring. Figure 2 shows front and rear views of the instrumentation interface box attached to the wired slab.

Testing Procedures

After the slabs were cast and cured, the upper surface of all slabs (except numbers 212 and 217) were ponded with a 3-percent sodium chloride solution until a corrosion current developed in a control slab. This ponding period lasted 46 days. After that time, the ponding was discontinued and the slabs exposed to the natural climate conditions of the Washington, D.C.-Northern Virginia area.

Data were usually gathered twice a month in the beginning and once a month later on. Data collection was usually completed within I week. Measurements were halted in case of rain and resumed when the excess water on the slab evaporated and the surface appeared dry. At the instrumentation box attached to the front of the slabs, the following data were collected with the coupling switch at the "on" position (bottom and top mat connected) in this sequence:

(a) Thermocouple readings for monitoring *temperature.*

(b) The voltage drop across a standard 0.5-ohm precision resistor. This measured voltage was then converted to the actual amount of *current flow* between the bottom and top mats.

The next set of slab readings were made with the switch at the "off" position:

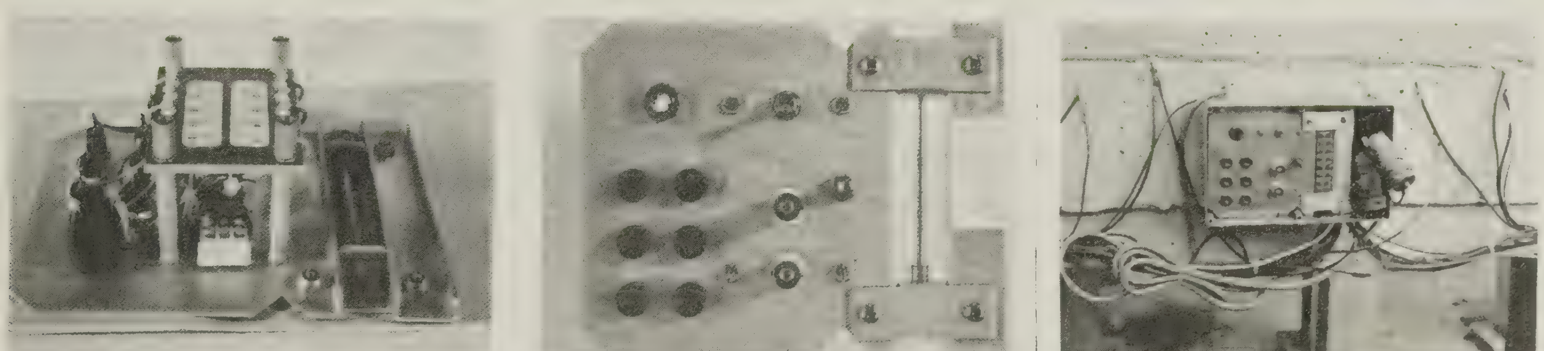
(c) The potential difference between the top and bottom rebar mats to measure the *driving voltage* of the corrosion cell (measured instantly after the switch was turned off).

(d) The *electrical potential* between the top mat and a copper/copper sulfate (CSE) reference half-cell measured on the top concrete slab surface at three marked positions.

(e) Same as (d) except that the CSE reference half-cell was placed at the bottom of the concrete slab surface for measuring bottom mat *electrical potential.*

(f) The *electrical resistance* between top and bottom mats using a 1,000-Hz AC meter.

The corrosion current measurement (b) was completed first (switch on) since uncoupling (switch off) causes rapid depolarization of corrosion cells, yielding inaccurate current flow readings. Once depolarization occurs, it takes a while to restore a steady-state polarization condition. Measurement (c) was taken instantly after uncoupling followed by measurements (d) and (e).



rear *front* *front after wiring*

Figure 2.—Instrumentation interface box.

Discussion and Interpretation

Salient features of the collected data are discussed below. The details for calculation and presentation of the measured data over a 7-year testing period for a typical reinforced concrete slab has been discussed in an earlier FHWA publication. (7)

Corrosion currents

Table 2 summarizes the corrosion data for the 18 instrumented slabs under test. Details on the properties of individual concrete slabs and fabrication are given elsewhere. (5) Table 3 contains the average weighted 70 °F (21 °C) corrosion currents for four control slabs (211, 213, 214, and 215 without calcium nitrite) and the average measured chloride content at the top rebar mat level. The table also contains data from two other control slabs (designated as 202 and 234) fabricated for an epoxy-coated rebar study. These slabs had black steel top and bottom mats, no calcium nitrite, and contained 15.6 lb Cl^-/yd^3 (9.36 kg Cl^-/m^3) of concrete. Figure 3 shows

Table 2.—Calcium nitrite and control slabs (black steel) at 2,429 days coupled

Slab no.	2.75% calcium nitrite	lb Cl^-/yd^3 hardened concrete	Average driving voltage (mV)	Avg. 70 °F corrosion current (μA)	Weighted avg. 70 °F corrosion current (μA)	Avg. 70 °F mat-to-mat resistance (ohms)	Avg. 70 °F iron consumed (grams)
(A) Ponded slabs							
211	no	3.6	7	275	376	16.3	22.82
216	yes	3.7	0	2	2	11.2	0.09
213	no	7.9	34	1,984	2,296	13.3	139.19
214	no	6.9	36	2,036	2,050	13.2	124.28
218[a]	yes	8.4	3	145	231	12.4	14.02
219	yes	8.4	2	88	115	11.0	6.97
220	yes	11.7	3	95	112	12.4	6.77
221	yes	10.9	3	115	113	12.3	6.87
222	yes	13.4	6	241	216	15.3	13.11
223[b]	yes	14.3	27	444	476	59.1	28.85
224	yes	16.7	25	1,173	1,152	14.6	69.86
225	yes	16.3	41	2,187	2,117	13.0	128.34
226	yes	16.8	40	2,566	2,591	11.0	157.07
227	yes	20.2	31	1,912	1,877	10.9	113.83
215[c]	no	23.3	51	2,865	3,566	13.5	216.23
228[d]	yes	22.6	58	2,034	1,663	63.5	100.84
(B) Slabs not ponded							
212[b]	no	0.0	0	6	7	133.0	0.42
217	yes	0.0	0	0	0	13.0	0.0

[a]This slab may have contained nondetectable surface cracks (over rebar) before ponding.
[b]Resistance measurements indicate these slabs are at least partially debonded between the top and bottom lifts.
[c]This slab is severely cracked.
[d]This slab is debonded between the top and bottom lifts; it is also severely cracked.

1 lb/yd^3 = 0.6 kg/m^3

Table 3.—Summary of corrosion currents: control (black steel) slabs at 2,429 days coupled

Control slab no.	Average Cl^- at top rebar level (lb Cl^-/yd^3)	Weighted avg. 70 °F corrosion current (μA)
211	3.6	376
213, 214	7.5	2,173
202, 234	15.6	3,652
215[a]	23.3	3,566

[a]This slab was severely cracked; thus data for it have not been plotted in figure 3.

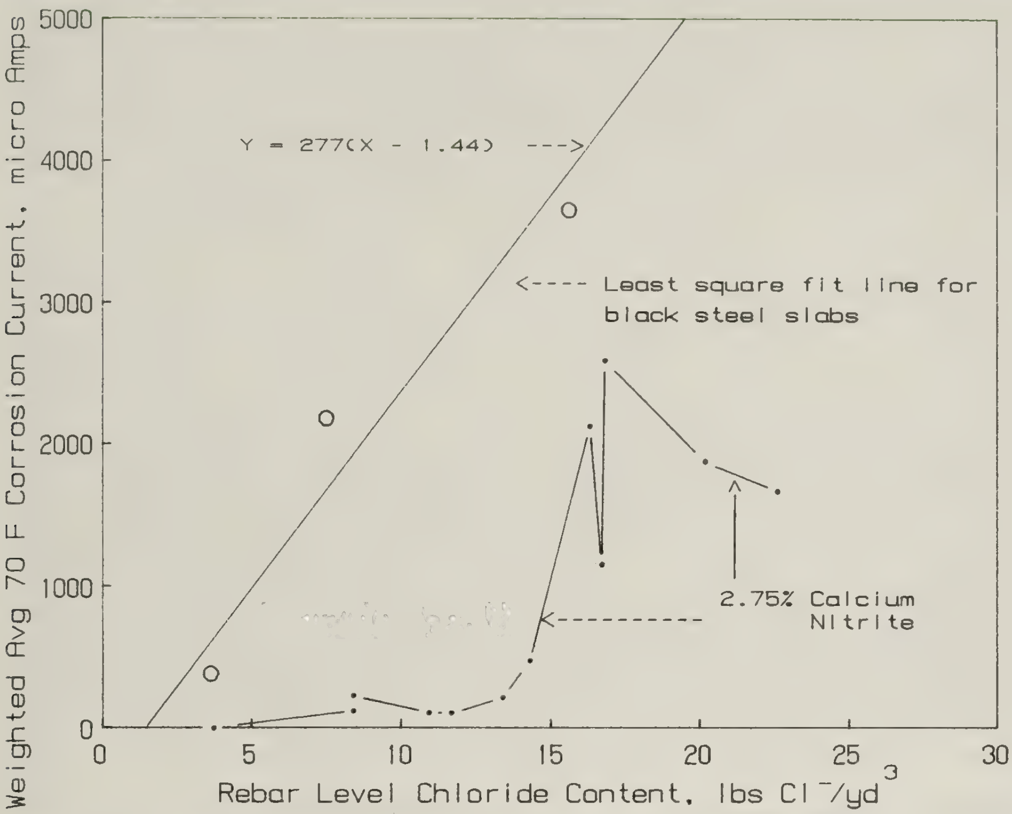


Figure 3.—Corrosion current versus chloride in concrete slabs at reinforcing steel level.

a plot of these data with a least-square fit line for this equation:

$$\text{Corrosion current (in } \mu\text{A)} = 277\,(\text{Cl}^- \text{ content in lb/yd}^3 - 1.44) \tag{1}$$

Also shown in figure 3 are the true weighted average corrosion current data for the calcium nitrite slabs versus average chloride content at the top rebar mat. Table 4 includes the derived corrosion currents from the least-square fit line in figure 3 for the slabs without calcium nitrite and compares it with the measured corrosion current data for slabs with calcium nitrite and known chloride content. The weighted average corrosion currents listed in tables 2, 3, and 4 were calculated by the following equation:

$$\text{weighted average corrosion current (in } \mu\text{A)} = \frac{\text{total cumulative amp x hr}}{\text{total days x } \frac{24 \text{ hr}}{1 \text{ day}}} \times \frac{10^6\ \mu\text{A}}{1 \text{ amp}} \tag{2}$$

Previous research on black steel rebars without calcium nitrite has shown that a strong macroscopic corrosion cell will develop between the top mat of reinforcing steel in chloride-contaminated concrete and the bottom mat of steel in chloride-free concrete. The data in table 4 and figure 3 show that black steel reinforced concrete slabs containing up to 2.75-percent calcium nitrite at the top mat have developed a macroscopic corrosion cell of varying strength between the top and bottom mats, depending upon the amount of chloride ions at the top mat rebar level. Qualitatively, it appears that the rate of corrosion (as measured by the corrosion current value) increases as the ratio of chloride to nitrite (Cl^-/NO_2) increases.

This trend is observed for slabs 216 to 228 with some exceptions. The anomalous results on a few slabs may be due to the debonding of bottom and top concrete lifts and excessive corrosion associated with concrete cracking at the top slab surface. On reviewing the corrosion current data in table 4, it appears that for Cl^-/NO_2 ranging between 0.29 to 1.11, reductions in the corrosion rate by an approximate factor of 10 are achieved. In other words, it would require 10 years to consume the same amount of iron as is consumed in 1 year by reinforced concrete slabs without calcium nitrite.

Moreover, table 4 shows that a Cl^-/NO_2 above 1.11 reduces the corrosion rate only by a factor of 2. Therefore, for calcium nitrite to be effective as a corrosion inhibitor (based on corrosion current data only), the Cl^-/NO_2 should be equal to or less than 1.1. The use of calcium nitrite as a corrosion inhibitor for a Cl^-/NO_2 greater than 1 does not appear to be significantly beneficial.

Half-cell potentials

Table 5 contains the summary of the data on half-cell potentials (measured with reference to the CSE half-cell) for the top and bottom reinforcing mats. The last column in table 5 lists the average potential differences between the two mats. In general, slabs 216 to 221 with a Cl^-/NO_2 less than 1 have smaller potential differences between the two mats and ranges from a low of a few millivolts (mV) to a high of 35 mV. In contrast, slabs 211, 213, and 214—which contain no calcium nitrite but have similar amounts of chloride to slabs 216 to 221—have larger average potential differences between the top and bottom mats, indicating more corrosion. Slabs 224 to 228, with a Cl^-/NO_2 greater than 1.1, have potential differences in the range of -52 mV to -149 mV.

Table 4.—Summary of corrosion currents: nitrite slabs versus control (black steel) slabs at 2,429 days coupled

Nitrite slabs	Average Cl^- at top rebar level ($lb\ Cl^-/yd^3$) [a]	Theoretical ($Cl^-/\overline{NO_2}$)[b]	Weighted avg. 70 °F corrosion current (μA)		Ratio of currents for no nitrite/nitrite
			Nitrite slabs	No nitrite slabs[c]	
216	3.7	0.29	2	625	312 to 1
218, 219	8.4	0.67	173	1,926	11.1 to 1
220, 221	11.4	0.90	112	2,759	24.6 to 1
222, 223[d]	14.0	1.11	346	3,479	10.1 to 1
224, 225	16.4	1.30	1,635	4,144	2.5 to 1
226, 227	18.5	1.47	2,234	4,726	2.1 to 1
228[d]	22.6	1.79	1,663	5,861	3.5 to 1

[a]Experimental values of average Cl^- are from table 6 in reference (*6*).
[b]Values of Cl^- to $\overline{NO_2}$ ratio are from table 9 in reference (*6*). The nitrite ions are derived from the calcium nitrite admixture in the fresh concrete.
[c]Values for weighted average corrosion current for slabs without nitrite were obtained using the least square-fit line in figure 3.
[d]Resistance measurements indicate that these slabs are partially debonded between the top and bottom lifts. Also, slab 228 is severely cracked; therefore, the measured weighted average corrosion current value may not be accurate.

These higher values for potential differences are indicative of larger corrosion macrocells compared with slabs with a $Cl^-/\overline{NO_2}$ less than 1. The potential difference values between the two reinforcing mats are approximately proportional to the strength of galvanic cells formed due to differential chloride concentration in two lifts and are somewhat analogous to the driving voltage values reported in table 2.

It is incorrect to interpret the absolute measured half-cell potential values for the top mat (embedded in chloride-contaminated concrete) and the bottom mat (embedded in salt-free concrete) according to the American Society for Testing and Materials (ASTM) C 876-80 Standard, since both the mats are polarized (coupled) until the measurement time. The half-cell potential values for corrosion, uncertain condition, and no corrosion as quoted in the ASTM C 876-80 Standard cannot be fully correlated with the measured values in table 5. However, it is quite apparent from the half-cell potential difference data between the two mats that calcium nitrite is an effective corrosion inhibitor for a $Cl^-/\overline{NO_2}$ of 1 or less in the hardened concrete.

Resistivity measurements

The data in table 6 show a wide scatter of average concrete resistivity values for the period 1980 to 1986 among different slabs: there are no significant differences between the slabs containing calcium nitrite and the control slabs. It is also apparent from table 6 that neither calcium nitrite nor chloride contents have significant influence on resistivity values. For most of the slabs, there is an increase in resistivity values for 1986 as compared with 1980. This increase over time may be due to:

- Drying up of interior concrete (since the measured resistance is for the interior concrete).
- Accumulation of corrosion byproducts in a few slabs with high chloride content.
- Partial debonding between the two lifts due to construction deficiencies at the time of slab fabrication or thereafter.

For slabs 212 and 228, a marked increase (up to tenfold) in resistivity indicates that these two slabs have been debonded. This debonding affects most of the nondestructive measurements such as driving voltage, corrosion current, and average potential differences between the top and bottom mats.

Visual survey

The condition of selected slabs as photographed in June 1987 is shown in figure 4; a summary of the visual survey data is presented in table 7. From the table, it is apparent that the slabs containing chloride (in the range of 6.9 to 23.3 $lb\ Cl^-/yd^3$ [4.14 to 13.98 $kg\ Cl^-/m^3$]) but no calcium nitrite have developed significant cracking (slabs 202, 213, 214, 215, and 234) and that rust products are leaching out of those cracks.

On the other hand, slabs with varying chloride levels and fixed levels of calcium nitrite (up to a $Cl^-/\overline{NO_2}$ of 0.90) did not show any visible cracking or rust spots on the concrete surface (slabs 216 to 221 except slab 218). These slabs had chloride concentrations up to 11.7 $lb\ Cl^-/yd^3$ (7.02 $kg\ Cl^-/m^3$) of concrete. Slab 222, which contained

Table 5.—Average electrical half-cell (black steel slabs) potentials

Slab no.	Variable[a]	Average electrical potential (mV, CSE) Top mat	Bottom mat	Average potential difference
211	No calcium nitrite: 3.6 lb Cl^-/yd^3	–111	–76	–35
212	No calcium nitrite: no chloride; no ponding	–154	–38	–116
213	No calcium nitrite: 7.9 lb Cl^-/yd^3	–219	–145	–74
214	No calcium nitrite: 6.9 lb Cl^-/yd^3	–300	–156	–144
215	No calcium nitrite: 23.3 lb Cl^-/yd^3	–393	–289	–104
216	2.75% calcium nitrite: 3.7 lb Cl^-/yd^3	–105	–106	+1
217	2.75% calcium nitrite: no chloride; no ponding	–117	–96	–21
218	2.75% calcium nitrite: 8.4 lb Cl^-/yd^3	–185	–174	–11
219	2.75% calcium nitrite: 8.4 lb Cl^-/yd^3	–174	–191	+17
220	2.75% calcium nitrite: 11.7 lb Cl^-/yd^3	–215	–180	–35
221	2.75% calcium nitrite: 10.9 lb Cl^-/yd^3	–181	–189	+8
222	2.75% calcium nitrite: 13.4 lb Cl^-/yd^3	–297	–245	–52
223	2.75% calcium nitrite: 14.3 lb Cl^-/yd^3	–369	–220	–149
224	2.75% calcium nitrite: 16.7 lb Cl^-/yd^3	–370	–284	–86
225	2.75% calcium nitrite: 16.3 lb Cl^-/yd^3	–406	–285	–121
226	2.75% calcium nitrite: 16.8 lb Cl^-/yd^3	–408	–297	–111
227	2.75% calcium nitrite: 20.2 lb Cl^-/yd^3	–418	–291	–127
228	2.75% calcium nitrite: 22.6 lb Cl^-/yd^3	–411	–300	–111

[a]Measured average chloride values between 1/2- to 2-in (1.27- to 5.08-cm) depth from the top surface of the slabs. This chloride was either added during the mixing of the top lift concrete, and/or the upper concrete slab surface was ponded with 3-percent salt solution.

Table 6.—Average resistance and resistivity

Slab no.	Variable[a]	Measurement period		Average 70 °F mat-to-mat resistance (ohms)	Average resistivity (ohms-cm)
211	No calcium nitrite: 3.6 lb Cl^-/yd^3	year	1980	10.61	7,103
			1983	16.84	11,890
			1986	20.84	14,715
			1980-86	16.30	11,509
212[b]	No calcium nitrite: no chloride; no ponding	year	1980	23.36	16,494
			1983	111.6	78,828
			1986	209.2	147,695
			1880-86	133.2	94,037
213	No calcium nitrite: 7.9 lb Cl^-/yd^3	year	1980	10.20	7,203
			1983	14.37	8,946
			1986	13.80	9,744
			1980-86	13.31	9,399
214	No calcium nitrite: 6.9 lb Cl^-/yd^3	year	1980	10.84	7,654
			1983	15.30	10,803
			1986	12.45	9,078
			1980-86	13.21	9,327
215	No calcium nitrite: 23.3 lb Cl^-/yd^3	year	1980	9.11	6,432
			1983	11.53	8,141
			1986	17.65	12,462
			1980-86	13.46	9,584
216	2.75% calcium nitrite: 3.7 lb Cl^-/yd^3	year	1980	7.14	5,042
			1983	11.44	8,078
			1986	13.91	9,821
			1980-86	11.20	7,909
217	2.75% calcium nitrite: no chloride; no ponding	year	1980	8.56	6,044
			1983	11.85	8,368
			1986	16.61	11,728
			1980-86	12.97	9,158
218	2.75% calcium nitrite: 8.4 lb Cl^-/yd^3	year	1980	8.10	5,719
			1983	12.71	8,975
			1986	16.04	11,326
			1980-86	13.06	9,222
219	2.75% calcium nitrite: 8.4 lb Cl^-/yd^3	year	1980	7.10	5,012
			1983	10.73	7,575
			1986	13.51	9,538
			1980-86	10.97	7,745
220	2.75% calcium nitrite: 11.7 lb Cl^-/yd^3	year	1980	7.80	5,507
			1983	12.28	8,671
			1986	15.47	10,923
			1980-86	12.36	8,728
221	2.75% calcium nitrite: 10.9 lb Cl^-/yd^3	year	1980	7.71	5,444
			1983	12.07	8,522
			1986	16.10	11,368
			1980-86	12.32	8,699

222	2.75% calcium nitrite: 13.4 lb Cl^-/yd^3	year	1980	8.86	6,256
			1983	14.70	10,379
			1986	20.73	14,637
			1980-86	18.11	12,787
223[b]	2.75% calcium nitrite: 14.3 lb Cl^-/yd^3	year	1980	27.01	19,072
			1983	84.96	59,989
			1986	29.98	21,169
			1980-86	58.37	41,214
224	2.75% calcium nitrite: 16.7 lb Cl^-/yd^3	year	1980	10.39	7,336
			1983	15.57	10,994
			1986	15.47	10,924
			1980-86	14.83	10,471
225	2.75% calcium nitrite: 16.3 lb Cl^-/yd^3	year	1980	10.39	7,336
			1983	15.05	10,626
			1986	11.74	8,290
			1980-86	14.51	10,245
226	2.75% calcium nitrite: 16.8 lb Cl^-/yd^3	year	1980	7.90	5,578
			1983	12.44	8,784
			1986	11.19	7,901
			1980-86	11.02	7,781
227	2.75% calcium nitrite: 20.2 lb Cl^-/yd^3	year	1980	7.63	5,387
			1983	12.11	8,551
			1986	10.74	7,584
			1980-86	11.05	7,802
228[b]	2.75% calcium nitrite: 22.6 lb Cl^-/yd^3	year	1980	8.49	5,995
			1983	10.85	7,662
			1986	281.9	199,019
			1980-86	81.29	57,398

[a]Measured average chloride values between 1/2- to 2-in (1.27- to 5.08-cm) depth from the top surface of the slabs. This chloride was either added during the mixing of the top lift concrete and/or upper concrete slab surface was ponded with 3-percent salt solution.
[b]Partially debonded slabs between the top and bottom lifts.

13.4 lb Cl^-/yd^3 (8.16 kg Cl^-/m^3) ($Cl^-/N\overline{O}_2$ = 1.06), had a few fine cracks and two rust spots. Similarly slab 223, containing $Cl^-/N\overline{O}_2$ of 1.13, has a moderate number of cracks, and the rust is coming out of these cracks. For slabs 224 to 228 with a Cl-/$N\overline{O}_2$ of 1.30 to 1.79, there is a large number of wide cracks and major rusting.

It is quite apparent that calcium nitrite is passive toward black reinforcing steel up to a $Cl^-/N\overline{O}_2$ of 0.90. The passive film breaks down when this ratio exceeds 0.90.

Conclusions

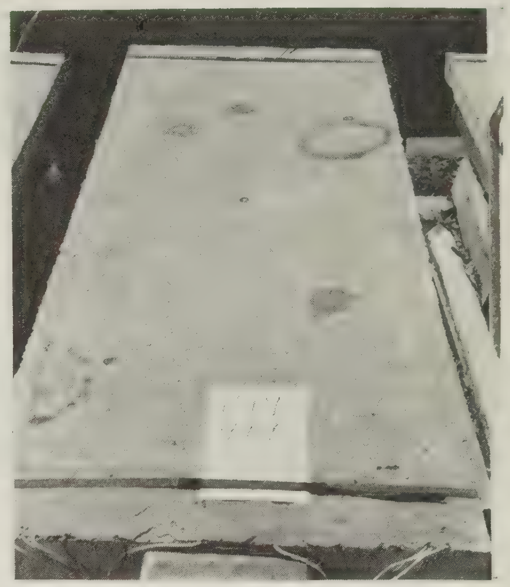
Tests of 18 reinforced concrete slabs over 7 years to determine the effectiveness of calcium nitrite as a corrosion-inhibiting admixture has led to the following conclusions. These conclusions are based on the periodic nondestructive measurements for corrosion current, half-cell potential, driving voltage, and resistivity as well as a visual survey at the end of the 7 years.

• In general, the nondestructive corrosion measurement techniques and the collected data correlate well with the visual survey of the concrete slabs under test. These nondestructive techniques offer a useful approach for monitoring the effectiveness of various corrosion-inhibiting admixtures and other additives when chloride-contaminated concrete may not show any visible distress or rust on the surface at an early age. (*6,7*)

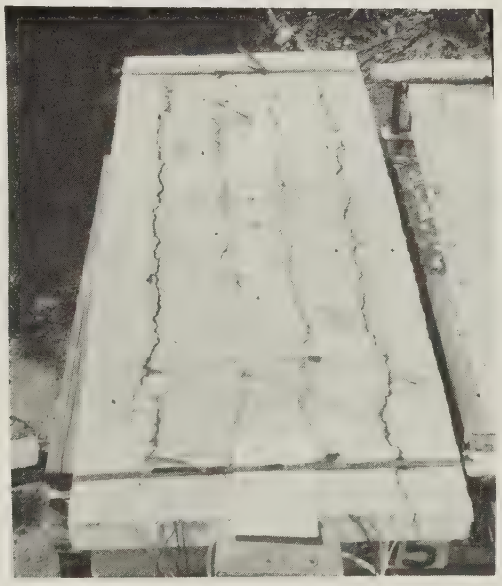
• The magnitude of corrosion current flow between the mats in salt-contaminated concrete and salt-free concrete, measured periodically, can be used semiquantitatively to monitor the performance of corrosion protection materials. Other measured parameters—such as driving voltage, half-cell potential, and concrete resistivity—provide additional useful data for proper interpretation of corrosion current flow data.

• The use of calcium nitrite was effective in reducing the rate of corrosion for black reinforcing steel embedded in poor-quality salt-contaminated concrete up to a Cl^-/NO_2^- of 0.90. This conclusion is based on both periodic nondestructive data collected over 7 years and a visual survey of the slabs. Although slabs fabricated with a higher Cl^-/NO_2^- (up to 1.11) indicated that there is a reduction in corrosion current flow by a factor of 10, the visual survey showed some cracking and rust spots on the concrete surface. Although slabs with a Cl^-/NO_2^- from 1.3 to 1.8 showed an average reduction in the corrosion current by a factor of 2.7, they did have cracks and major rusting out of cracks and on the surrounding concrete surface. In addition, there were some hollow and spalled areas present on these slabs (224 to 228).

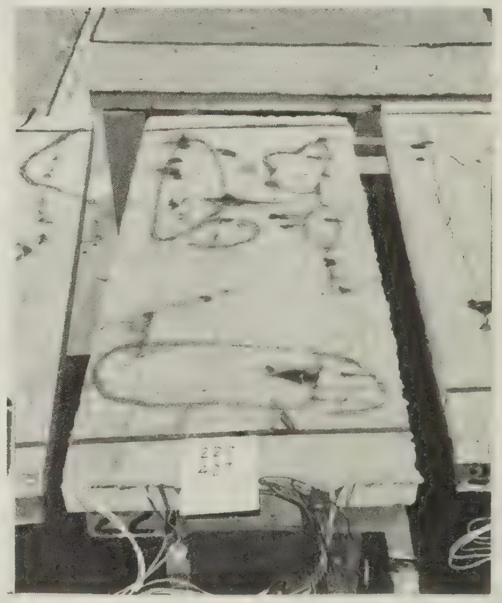
slab 213: No calcium nitrite: 7.9 lb Cl^-/yd^3



slab 219: 2.75% calcium nitrite: 8.4 lb Cl^-/yd^3



slab 215: No calcium nitrite: 23.3 lb Cl^-/yd^3



slab 227: 2.75% calcium nitrite: 20.2 lb Cl^-/yd^3

Figure 4.—Condition of selected slabs at 2429 days coupled (June 1987).

• Calcium nitrite appears to be effective because it does not allow a large electrical potential difference to develop between adjoining steel in the top mat or between the top and bottom mats. In the absence of calcium nitrite, there would be cathodic and anodic areas which would allow the generation of corrosion current and the dissolution of iron from the top mat in chloride-contaminated concrete.

Table 7.—Visual survey summary

Slab no.	Calcium nitrite[a]	lb Cl^-/yd^3	Comments
211	0.0	3.6	No cracks; no rust
212	0.0	0.0	No cracks; no rust
213	0.0	7.9	Moderate amounts of wide cracks; some rust
214	0.0	6.9	Small to medium cracks; no rust
215	0.0	23.3	Large amounts of wide cracks; major rust
216	2.75	3.7	No cracks; no rust
217	2.75	0.0	No cracks; no rust
218[b]	2.75	8.4	Fine cracks; rust visible
219	2.75	8.4	No cracks; no rust
220	2.75	11.7	No cracks; one rust spot
221	2.75	10.9	Fine cracks; two rust spots
222	2.75	13.4	Cracks; rust coming from cracks
223	2.75	14.3	Wide cracks; spalls; rust coming from cracks
224	2.75	16.7	Wide cracks; spalls; major rust
225	2.75	16.3	Wide cracks; spalls; major rust
226	2.75	16.8	Wide cracks; spalls; major rust
227	2.75	20.2	Wide cracks; spalls; major rust
228	2.75	22.6	Wide cracks; spalls; major rust

[a]Percentage by weight of cement
[b]This slab may have contained surface cracks (over rebar) before ponding.

Recommendation

If chosen as the corrosion protection system, sufficient quantities of calcium nitrite should be added to the fresh concrete to provide a Cl^-/NO_2^- of less than 1 at the steel level closest to the exposed concrete surface during the structure's expected design life. This chloride level is the expected accumulative value attained at the steel level due to the exposure of the concrete surface to deicing marine salts. This recommendation is based on the results from the limited number of slabs tested in this study where most of the chloride ions were added along with nitrite ions at the time of construction. Here both the ions were engaged simultaneously in complicated competing reactions: corrosion versus passivation at the reinforcing steel surface. It is reasonable to assume that, for a normal situation in which chloride ions are penetrating more slowly into hardened concrete, Cl^-/NO_2^- higher than 1 might be tolerable without undue corrosion.

A detailed research study is needed to determine the long-term stability and availability of sufficient nitrite ions since they are required only when the chloride ions have penetrated through the hardened concrete surface and started corroding the reinforcing steel.

Studies have indicated that the period for accumulation of sufficient chloride ions (above the threshold level for corrosion initiation) through 2 in (5.08 cm) of good quality concrete cover may be 7 to 12 years. On the other hand, poor-quality concrete can accumulate adequate chloride (1.25 lb Cl^-/yd^3 [0.75 kg Cl^-/m^3] at a 1-in (2.5 cm) depth through 7 deicing salt applications. (*8*)

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Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Federal Highway Administration, Office of Research, Development, and Technology (RD&T). The Office of Engineering and Highway Operations Research and Development (R&D) includes the Structures Division, Pavements Division, and Materials Division. The Office of Safety and Traffic Operations R&D includes the Traffic Systems Division, Safety Design Division, and Traffic Safety Research Division. All reports are available from the National Technical Information Service (NTIS). In some cases, limited copies of reports are available from the RD&T Report Center.

When ordering from the NTIS, include the PB number (or the publication number) and the publication title. Address requests to:

National Technical Information Service
5285 Port Royal Road
Springfield, Virginia 22161

Requests for items available from the RD&T Report Center should be addressed to:

Federal Highway Administration
RD&T Report Center, HRD-11
6300 Georgetown Pike
McLean, Virginia 22101-2296
Telephone: (703) 285-2144

Development of Procedures for the Calibration of Profilographs, Publication No. FHWA-RD-89-110

by Pavements Division

A recent review of the equipment and methods used to measure the roughness of new pavements revealed that most States use the California profilograph with acceptance limits based on rideability criteria. Bump specifications—used throughout the highway construction industry for many years—are useful in controlling individual vertical deviations of pavement profile, but do not provide information on the overall pavement roughness.

A full-scale testing program investigated the basic roughness characteristics of new pavements. These characteristics are represented by power spectral density functions which are then used to generate average profile data. Road roughness measuring devices, including the California, Rainhart, and Ames profilographs, a profilometer, and a Mays ride meter were evaluated for: frequency response, precision, repeatability, reliability, and ease of operation. Correlation coefficients between the different roughness measuring devices were calculated. The correlation for both the California and Rainhart profilographs, with the profile calculated roughness index, improved when a continuous roughness scale was used.

The effect of varied design parameters on profilograph performance was studied using computer simulations.

Based on these results, specifications for an improved profilograph are given. Such a profilograph should have a better frequency response over the roughness range of approximately 1.6 to 32 ft (0.5 to 10 m) wavelengths. However, with acceptance criteria of less than approximately 7 in/mi (0.1 m/1 m)—the most widely used current criteria—the utility of any profilograph becomes questionable when it is to be used with a bonus and penalty payment system.

Finally, a two-step procedure for calibrating profilographs is given. Step one uses a manufactured calibration surface and the comparison is made to an ideal, simulated profilograph of similar construction. Step two is used to establish the calibration equations for a given profilograph model and needs to be repeated only after major repairs or modifications.

This publication may only be purchased from the NTIS. (PB No. 90-150889/AS, Price code: A10.)

Trade-Off Between Delineation and Lighting on Freeway Interchanges, Publication No. FHWA-RD-88-223

by Traffic Safety Research Division

This report documents the methodology and the results of a study to determine whether driver performance at partially lighted interchanges could be improved by upgrading the delineation system to equal performance at fully illuminated interchanges.

The study was carried out under dry and rainy weather conditions. The investigation evaluated drivers' ramp speeds, lateral placement, edgeline and gore encroachments, brake activation, and use of high beams. As part of the study, the effects of transient visual adaptation (TVA) were investigated. TVA is a temporary reduction in the sensitivity of the eye when a person moves from a bright into a darker area. Target detection distance and driver ramp speed performance downstream from the partial lighting showed such an effect was occurring.

The study results also show that even with a substantial upgrade of delineation, driver performance under partial lighting will not equal that of full lighting.

This publication may only be purchased from the NTIS. (PB No. 90-156936/AS, Price code: A06.)

Pavement Friction Measurement Normalized for Operational, Seasonal, and Weather Effects, Publication No. FHWA–RD–88–069

by Pavements Division

This report describes the validation of models previously developed to normalize pavement friction measurements to standard environmental and operational conditions. Improvements to the techniques and equipment for pavement friction measurement are recommended.

Two models were examined for normalization of weather-related and seasonal variation of pavement friction. Their performance was found to be comparable. However, the generalized prediction model is more readily applied than the mechanistic model which requires knowledge of the percent normalized gradient of the pavements to be adjusted. The suggested procedure for normalization requires that three pavements of each class in a homogeneous climate be tested once a month during the testing season. The data then can be used to develop the coefficients in the model to adjust single measurements of all other pavements in that class and climate to standard conditions.

Recommendations for improving friction measurement equipment and techniques include:

- Use current equipment with both smooth-treaded and ribbed standard test tires.

- Modify the current equipment to measure the acceleration of the test tire during spinup after being locked as the indication of pavement friction. This will provide simpler instrumentation by making it possible to eliminate the force transducer.

This publication may only be purchased from the NTIS. (PB No. 90-149865/AS, Price code: A10.)

Rehabilitation of Concrete Pavements:

Vol. I: Repair Rehabilitation Techniques, Publication No. FHWA–RD–88–071

Vol. II: Overlay Rehabilitation Techniques, Publication No. FHWA–RD–88–072

Vol. III: Evaluation and Rehabilitation System, Publication No. FHWA–RD–88–073

Vol. IV: Appendixes, Publication No. FHWA–RD–88–074

by Pavements Division

Extensive field, laboratory, and analytical studies were conducted on the evaluation and rehabilitation of concrete pavements. Field studies included over 350 rehabilitated pavement sections throughout the U.S., and the construction of two field experiments. A laboratory study was conducted on anchoring dowels in full-depth repairs. Analyses of field and laboratory data identified performance characteristics, improved design and construction procedures, and provided deterioration models for rehabilitated pavements. A concrete pavement advisory system was developed to assist engineers in project level evaluation and rehabilitation.

The repair techniques in volume I include full-depth repair, partial-depth repair, load transfer restoration, edge support, and diamond grinding.

The overlay techniques in volume II include bonded concrete, unbonded concrete, and crack and seat with an asphalt concrete overlay.

Volume III presents a comprehensive concrete pavement evaluation and rehabilitation advisory system for jointed plain, jointed reinforced, and continuously reinforced concrete pavements.

Volume IV contains a description of the data collection procedures, original pavement and rehabilitation design factors, extent of the data base, description of data base variables, and documentation of the laboratory dowel anchoring experiment.

These publications may only be purchased from the NTIS.

Vol. I (PB No. 90-149725/AS, Price code: A12.)

Vol. II (PB No. 90-154568/AS, Price code: A09.)

Vol. III (PB No. 90-149733/AS, Price code: A19.)

Vol. IV (PB No. 90-149741/AS, Price code: A06.)

Implementation/User Items "how-to-do-it"

The following are brief descriptions of selected items that have been completed recently by State and Federal highway units in cooperation with the Office of Implementation, Office of Research, Development, and Technology (RD&T), Federal Highway Administration. Some items by others are included when the they are of special interest to highway agencies. All reports are available from the National Technical Information Service (NTIS). In some cases, limited copies of reports are available from the RD&T Report Center.

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RD&T Report Center, HRD–11
6300 Georgetown Pike
McLean, Virginia 22101–2296
Telephone: (703) 285–2144

New Methods for Determining Requirements for Truck-Climbing Lanes, Publication No. FHWA–IP–89–022

by Office of Implementation

Highway design engineers have constructed hill-climbing lanes to facilitate traffic flow and to improve highway safety upgrades. A recent Federal Highway Administration (FHWA)-sponsored study found that current guidelines may lead designers to add and maintain unnecessary hill-climbing lanes.

The report highlights four major findings on the FHWA study:

- Since current design guidelines are conservative, critical lengths of grade for single-unit trucks and tractor-semitrailers are shorter than needed.
- Single-unit trucks and tractor-semitrailers perform better than single-unit trucks with trailers and doubles. The improved performance matches current guidelines.
- Critical length of grade should be based on the weight-to-available power ratio of current truck mix, rather than on assumptions about performance.
- Highway designers need comprehensive methods to determine the need for hill-climbing lanes.

This report also presents tools developed in the study for designing hill-climbing lanes and is based on "Methods for Predicting Truck Speed Loss on Grades—Final Technical Report" FHWA/RD–86/059.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-156928/AS, Price code: A03.)

Improving Truck Safety at Interchanges, Publication No. FHWA–IP–89–024

by Office of Implementation

The interaction between truck dynamics and interchange geometry can contribute to rollovers, jackknives, and other loss-of-control accidents. Therefore, this report offers highway engineers guidance in designing interchanges to reduce truck accidents. The research was conducted by the University of Michigan Transportation Research Institute (UMTRI) and supported by the Federal Highway Administration (FHWA).

Engineers can apply corrective actions to six specific ramp design features that were found to contribute to truck accidents: poor transitions to superelevation, abrupt changes in compound curves, short deceleration lanes preceding tight-radius exits, curbs placed on the outside of ramp curves, lowered friction levels on high-speed ramps, and substantial downgrades leading to tight ramp curves.

Countermeasures for these design problems include incorporating a greater safety margin into formulations for side friction factors, reviewing and modifying posted speed limits, improving curve condition and downgrade signs at interchanges, increasing deceleration lane length, overlaying curbs with wedges of pavement, or eliminating curbs altogether, resurfacing ramps with high-friction overlays, and redesigning sites where accidents are common.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-159278/AS, Price code: A03.)

Operating Larger Trucks on Roads with Restrictive Geometry: Summary Report, Publication No. FHWA–IP–89–025

by Office of Implementation

Changes in the 1982 Surface Transportation Assistance Act—allowing wider and longer trucks on the national network—have raised questions about highway safety. The Federal Highway Administration sponsored a study that investigated the performance of trucks of various lengths and widths on roads with restrictive geometry. This report highlights the main findings of that study for transportation officials and practicing engineers.

Field studies at both urban and rural sites indicated that truck drivers compensate for the reduced operating capabilities of larger trucks. Despite driver skill, however, trucks on urban roads encroached into other lanes on streets with widths less than 12 ft (3.65 m). Intersections with less than 60-ft (18.24-m) corner radii caused some problems for most truck types, especially those wider than 8.5 ft (2.58 m). Prohibiting large trucks from turning onto narrow urban streets, employing turn movement templates in roadway design, adjusting signal and/or left-turn lane lengths, and manufacturing 48-ft (14.59-m) semitrailers with only forward axles may minimize these and other problems.

On rural roads, lanes wider than 12 or 13 ft (3.65 or 3.95 m) allowed oncoming vehicles to move further right to avoid trucks, and shoulders wider than 4 ft (1.22 m) allowed oncoming vehicles a greater margin of safety. At sharp curves (7 to 15 degrees), opposing vehicles slowed downed significantly and made other undesirable changes to pass large trucks. Consideration should be given to reducing the sharpness of curves greater than 7 degrees and to allowing large trucks only on to two-lane rural roads with lanes at least 12 ft (3.65 m) wide and shoulders greater than 4 ft (1.22 m).

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-153669/AS, Price code: A03.)

Grade Severity Rating System, Publication No. FHWA-IP-88-015

by Office of Implementation

The purpose of the grade severity rating system (GSRS) is to reduce the probability of large truck runaways on severe downgrades. The GSRS is based on a mathematical model that uses gross truck weight and physical characteristics of the downgrade to predict the temperature of the truck's braking system. The brake temperature estimates are used to determine maximum safe descent speeds for different categories of truck weight. The maximum safe descent speed information is presented to the truck drivers by weight specific speed signs (WSS) installed at the downgrade sites.

This user manual presents the procedural steps and analysis required to select appropriate downgrades, determine the safe downgrade speeds, and install the WSS signs. As a part of this project, determination of the safe downgrade speeds was accomplished by the GSRS computer program—developed for use on IBM and compatible personal computers. In addition to determining safe truck downgrade speeds, this program has applications that include how to:

- Determine the grade severity.
- Establish downhill truck speed limits.
- Establish the need for and location of truck escape ramps.
- Analyze truck accidents.

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. (PB No. 90-161423/AS, Price code: A06.)

Geotextile Specifications for Highway Applications, Publication No. FHWA–TS–89–026

by Office of Implementation

This report was prepared after a review of the most recent specifications on geotextiles and related geosynthetic materials from 46 of 52 (50 States, District of Columbia, and Puerto Rico) State agencies.

With a focus on required property values and individual State specification comparisons, the report continues with a discussion of a sample generic geotextile specification. Although detailed, it includes all features which are necessary for proper use of geotextiles in highway applications.

The sample specification does *not* recommend numeric product values in the "property requirements" section. Since numeric values are both geographic dependent and site specific, this is left to the individual State agency. Such values also reflect the philosophy of the specifying agency; ranging from extreme conservatism in required properties to extreme concern over costs.

This publication may only be purchased from the NTIS. (PB No. 90-1254109/AS, Price code: A05.)

Work Zone Traffic Management Synthesis:

Barrier Delineation Treatments Used in Work Zones, Publication No. FHWA–TS–89–033

Selection and Applications of Flashing Arrow Panels, Publication No. FHWA–TS–89–034

Work Zone Pedestrian Protection, Publication No. FHWA–TS–89–035

Use of Rumble Strips in Work Zones, Publication No. FHWA–TS–89–037

by Office of Implementation

Each of the above reports synthesizes research findings on one of the following current practices in work zones:

- Delineation of portable concrete safety-shaped barriers (CSSB).
- Selection and application of arrow panels.
- Control and protection of pedestrian traffic
- Design, selection, and application of rumble strips.

The information is based on a review of research reports and work zone manuals from a selection of State and city highway agencies, discussions with highway officials, and field observations of selected highway construction projects. These reports present an assessment of the state-of-the-practice and make recommendations for further research and future revisions of the *Manual on Uniform Traffic Control Devices.*

Limited copies of these publications are available from the RD&T Report Center. Copies may also be purchased from the NTIS.

FHWA–TS–89–033: (PB No. 90-124074/AS, Price code: A03.)
FHWA–TS–89–034: (PB No. 90-124082/AS, Price code: A05.)
FHWA–TS–89–035: (PB No. 90-124124/AS, Price code: A04.)
FHWA–TS–89–037: (PB No. 90-124090/AS, Price code: A04.)

Work Zone Traffic Management Synthesis: Tiedown Methods for Precast Concrete Safety Shaped Barriers, Publication No. FHWA–TS–89–036

by Office of Implementation

This report is a synthesis of research findings and current practices regarding the design and application of systems for anchoring (tieing down) portable concrete safety shaped barriers (CSSB) to highway pavements. The presented information is based on a literature review, field observation, and discussions with highway officials in selected States. The report presents design graphics on a number of methods for controlling sliding, tilting, and overturning of portable CSSB's, identifies areas for research, and makes recommendations regarding practices and information that should be included in the American Association of State Highway and Transportation Officials' *Roadside Design Guide.*

Limited copies of this publication are available from the RD&T Report Center. Copies may also be purchased from the NTIS. FHWA–TS–89–036: (PB No. 90-124132/AS, Price code: A04.)

New Research in Progress

The following new research studies reported by the FHWA's Office of Research, Development, and Technology are sponsored in whole or in part with Federal highway funds. For further details on a particular study, please note the kind of study at the end of each description:

- **FHWA Staff and Administrative Contract Research contact *Public Roads*.**
- **Highway Planning and Research (HP&R) contact the performing State highway or transportation department.**
- **National Cooperative Highway Research Program (NCHRP) contact the Program Director, NCHRP, Transportation Research Board, 2101 Constitution Avenue, NW, Washington, DC 20418.**
- **Strategic Highway Research Program (SHRP) contact the SHRP, 818 Connecticut Avenue, NW, 4th floor, Washington, DC 20006.**

NCP Category A—Highway Safety

A.3: Highway Safety Analysis

Title: Wet Weather Accident Analysis and Skid Resistance Data Management System (NCP No. 4A3B0142)
Objective: Develop a methodology that would effectively identify abnormal pavement sections in conjunction with skid resistance tests. Develop a computer information system with an integrated relational data base.
Performing Organization: Louisiana State University, Baton Rouge, LA 70803
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: December 1991
Estimated Cost: $164,685 (HP&R)

A.4: Special Highway Users

Title: Hazardous Materials Routing (NCP No. 4A4E1072)

for initiating a hazardous materials routing program at the State level. Establish criteria to select the safest highway routes for transporting hazardous materials in accordance with 49 CFR. Based on these criteria, select a working model to manage and process a viable routing pattern to safely and efficiently transport these materials.
Performing Organization: New Jersey Department of Transportation, Trenton, NJ 08623
Expected Completion Date: April 1992
Estimated Cost: $92,278 (HP&R)

NCP Category B—Traffic Operations

B.1: Traffic Management Systems

Title: Evaluate Vehicle Navigation and Communication Technologies (NCP No. 4B1D0072)
Objective: Develop a practical invehicle navigator with data logging capabilities for use by the California Department of Transportation. Demonstrate at least one position fixing (caging) technology.
Performing Organization: California Department of Transportation, Sacramento, CA 95807
Expected Completion Date: January 1992
Estimated Cost: $153,012 (HP&R)

NCP Category C—Pavements

C.3: Field and Laboratory Test Methods

Title: Equipment Management (NCP No. 4C3A1732)
Objective: Evaluate, correlate, and maintain pavement condition survey equipment: model 8300 roughness surveyer, face dipstick, Mays meter, and others.
Performing Organization: Louisiana Transportation Research Center, Baton Rouge, LA 70804
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: December 1992
Estimated Cost: $75,000 (HP&R)

C.4: Pavement Management Strategies

Title: Life-Cycle Cost Estimates for Pavements (NCP No. 4C4C2992)
Objective: Develop procedures for estimating pavement life-cycle costs for routine use by the Alabama Highway Department. Analyze American Association of State Highway and Transportation Officials' economic evaluation procedures for pavements. Identify data needs. Review available departmental data sources. Design a methodology applicable to the department for use on microcomputers. Develop procedures for data base organization and update cycles.
Performing Organization: University of Alabama, University, AL 35486
Funding Agency: Alabama State Highway Department
Expected Completion Date: December 1990
Estimated Cost: $86,086 (HP&R)

NCP Category D—Structures

D.1: Bridge Design

Title: Feasibility Evaluation of Utilizing High Strength Concrete in Design and Construction of Highway Bridge Structures (NCP No. 4D1C1062)
Objective: Examine and evaluate the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specification for Highway Bridges* in terms of concrete design strengths. Design, construct, and test full-size AASHTO type II girders. Design, construct, instrument, and monitor a prototype bridge depending on the test results of the type II girders. Information will serve as a guide toward future design and construction of high-strength concrete highway bridge structures.
Performing Organization: Tulane University, New Orleans, LA 70118
Funding Agency: Louisiana Department of Transportation and Development
Expected Completion Date: January 1995
Estimated Cost: $235,227 (HP&R)

Title: Behavior of Concrete Bridge Decks and Slabs Reinforced with Epoxy-Coated Steel (NCP No. 4D1B3182)
Objective: Examine the behavior of epoxy-coated rebars in bridge decks through literature search, field evaluation, and laboratory study. Field evaluation: map crack patterns and perform permeability tests on bridge decks and slabs reinforced with epoxy-coated steel. Laboratory study: test 40 reinforced concrete slab specimens in static and fatigue loadings to examine the bond strength and performance of splices.
Performing Organization: Purdue University, Lafayette, IN 47907
Funding Agency: Indiana Department of Transportation
Expected Completion Date: July 1992
Estimated Cost: $130,000 (HP&R)

D.3: Hydraulics and Hydrology

Title: Instrumentation for Measuring Scour at Bridge Piers and Abutments (NCP No. 5D3C1612)
Objective: Develop, test, and evaluate scour-monitoring devices that can be mounted to a bridge to indicate the maximum scour depth that occurs at piers and abutments during floods. Without attendance by survey crews, equipment must operate during a storm or a large flood. Instrumentation can be installed on most existing bridges or during construction of new bridges. The instrumentation will be economically and technically feasible for highway engineers to use in monitoring scour performance of bridges.
Performing Organization: Resource Consultants, Inc., Fort Collins, CO 80522
Expected Completion Date: March 1992
Estimated Cost: $280,209 (NCHRP)

D.4: Corrosion Protection

Title: Cathodic Protection Utility for Prestressed Concrete Pilings (NCP No. 4D4B2162)
Objective: Establish the criteria whereby salt-contaminated prestressed pilings may be cathodically protected. Characterize any tendon embrittlement tendency and loss of bond that may accompany cathodic protection of prestressed concrete.
Performing Organization: Florida Atlantic University, Boca Raton, FL 22431
Funding Agency: Florida Department of Transportation
Expected Completion Date: December 1990
Estimated Cost: $68,000 (HP&R)

NCP Category E—Materials and Operations

E.1: Asphalt and Asphaltic Mixtures

Title: Laboratory Determination of Resilient Modulus for Flexible Pavement Design (NCP No. 5E1B2171)

Objective: Develop laboratory testing procedures for determining the resilient moduli of component materials in flexible pavements accounting for varying field conditions. Compare field and laboratory moduli to determine the applicability of the tests. Evaluate the use and constraints of resilient moduli to establish layer coefficients for structural design purposes.

Performing Organization: Georgia Tech Research Institute, Atlanta, GA 30332

Expected Completion Date: September 1992

Estimated Cost: $425,000 (NCHRP)

E.2: Cement and Concrete

Title: Modulus of Rupture and Permeability of Structural Concrete (NCP No. 4E2D1043)

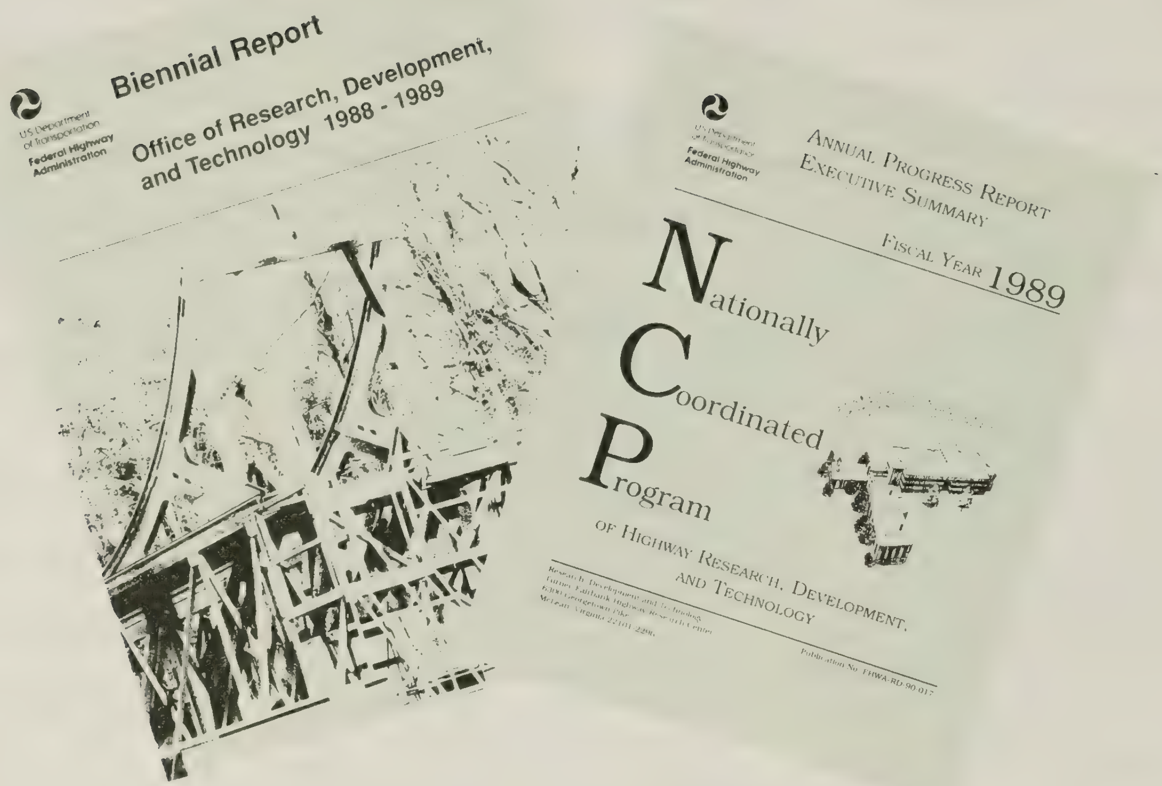
Objective: Determine the moduli of rupture and water permeability of Florida class II, III, and IV structural concrete made with five Florida aggregates and one out-of-State aggregate. Relate corrosion protection and concrete permeability using information in the literature. Develop guidelines for a concrete performance specification that addresses durability as well as concrete strength and workability.

Performing Organization: University of Florida, Gainsville, FL 32611

Funding Agency: Florida Department of Transportation

Expected Completion Date: December 1990

Estimated Cost: $118,081 (HP&R)



New Publications

Biennial Report: Office of Research, Development, and Technology 1988-1989, Publication No. FHWA-RD-90-007

The Office of Research, Development, and Technology (RD&T) has released its 1988-1989 biennial report. This report is a continuation of the series of annual and biennial reports published since 1974. It covers the period from October 1987 through September 1989.

The 1988-1989 report covers exclusively the offices of research, development, and technology housed at the Turner-Fairbank Highway Research Center. Along with an overview of the research center, this report includes summaries of the various research programs managed by the staff. The back section includes a list of publications for the biennial period.

While supplies last, individual copies of the reports in the series are available without charge from the Federal Highway Administration, RD&T Report Center, HRD-11, 6300 Georgetown Pike, McLean VA 22101-2296. Telephone: (703) 285-2144.

Nationally Coordinated Program of Highway Research, Development, and Technology: Annual Progress Report, Executive Summary, Fiscal Year 1989, Publication No. FHWA-RD-90-017

This executive summary gives an overview of progress being made under the Nationally Coordinated Program (NCP) of Highway Research, Development, and Technology during the period from October 1, 1988 through September 30, 1989. The Office of RD&T uses the NCP as a management framework for highway research activities. The objectives of the NCP are to:

- Ensure resource concentration on critical problems.
- Minimize duplicated efforts among researchers.
- Identify and highlight gaps in research.

This report covers technologies for highway design, construction, and operations including the specific categories of: Highway Safety, Traffic Operations, Pavements, Structures, Materials and Operations, Policy and Planning, and Motor Carrier Transportation.

Limited copies of this executive summary are available from the RD&T Report Center, HRD-11, 6300 Georgetown Pike, McLean VA 22101-2296. Telephone: (703) 285-2144.

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U.S. Department of Transportation

Federal Highway Administration

OCDE

OECD

Seminar on Technology Transfer and Adaptability in Industrialized Nations

The Organization for Economic Co-operation and Development (OECD) is sponsoring a Seminar on Technology Transfer and Adaptability in Industrialized Nations on November 11 through 14, 1990 in Orlando, Florida. The Federal Highway Administration will host the seminar for the OECD. Participants at the seminar will identify and document the principal technology transfer techniques, methods, practices, and costs as they currently exist in the OECD industrialized countries, and prepare recommendations for use by member countries that will promote the rapid flow of highway technology within and among countries. The seminar will be conducted in English, French, and Spanish with simultaneous interpretation.

Conference registration details and other information may be obtained from the conference coordinator:

Mr. George M. Shrieves
Director, National Highway Institute
Federal Highway Administration
6300 Georgetown Pike
McLean, Virginia 22101-2296
UNITED STATES

Telephone: (703) 285-2770
Facsimile: (703) 285-2379

A second seminar is being planned for 1991. This seminar will focus on the technology transfer needs of developing countries.

U.S. Department
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**Federal Highway
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