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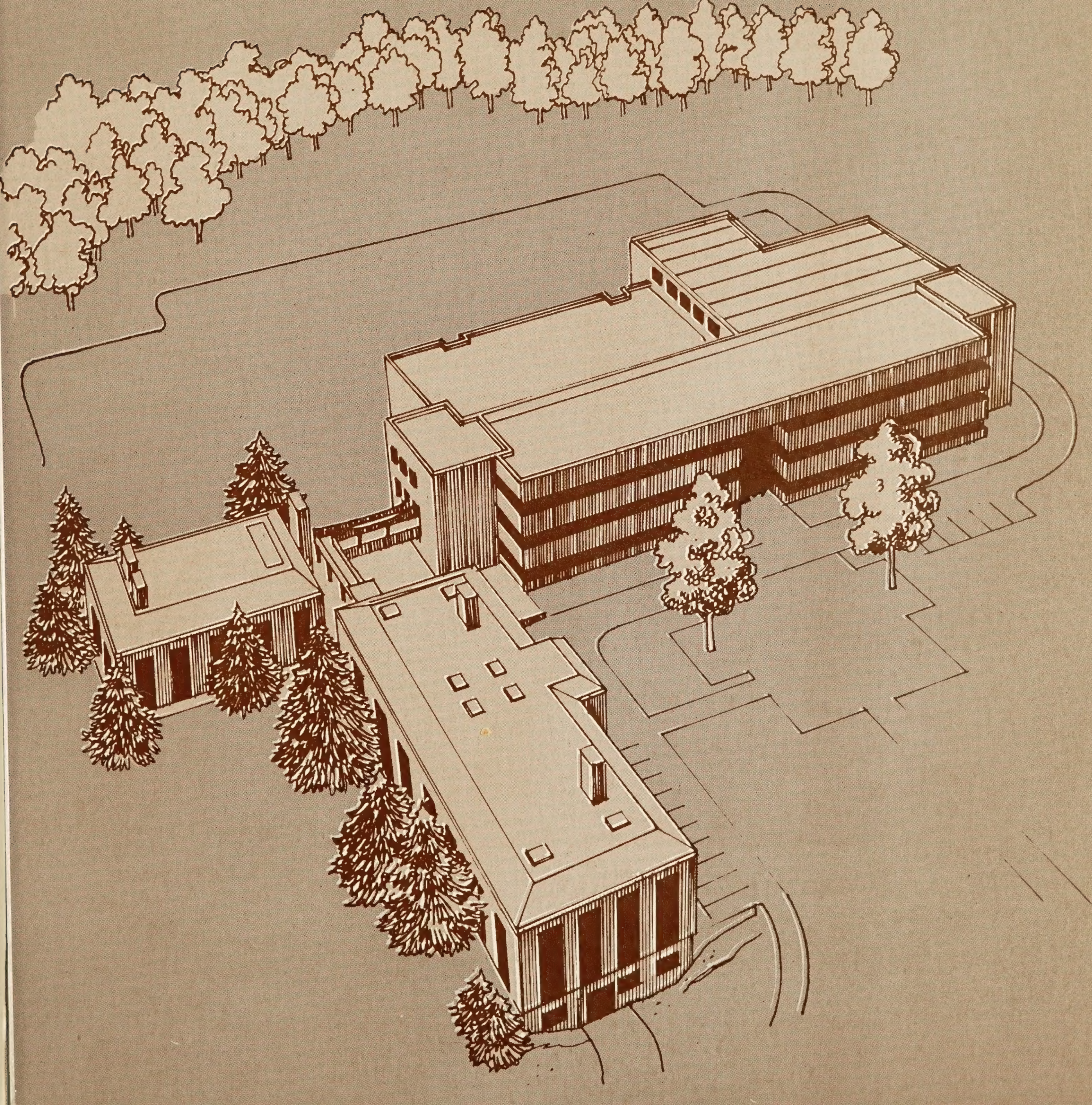

Public Roads

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COVER: The new Turner-Fairbank Highway Research Center in McLean, Va., was dedicated on May 5. An article on the new facility and the dedication ceremony will appear in the next issue of *Public Roads*.

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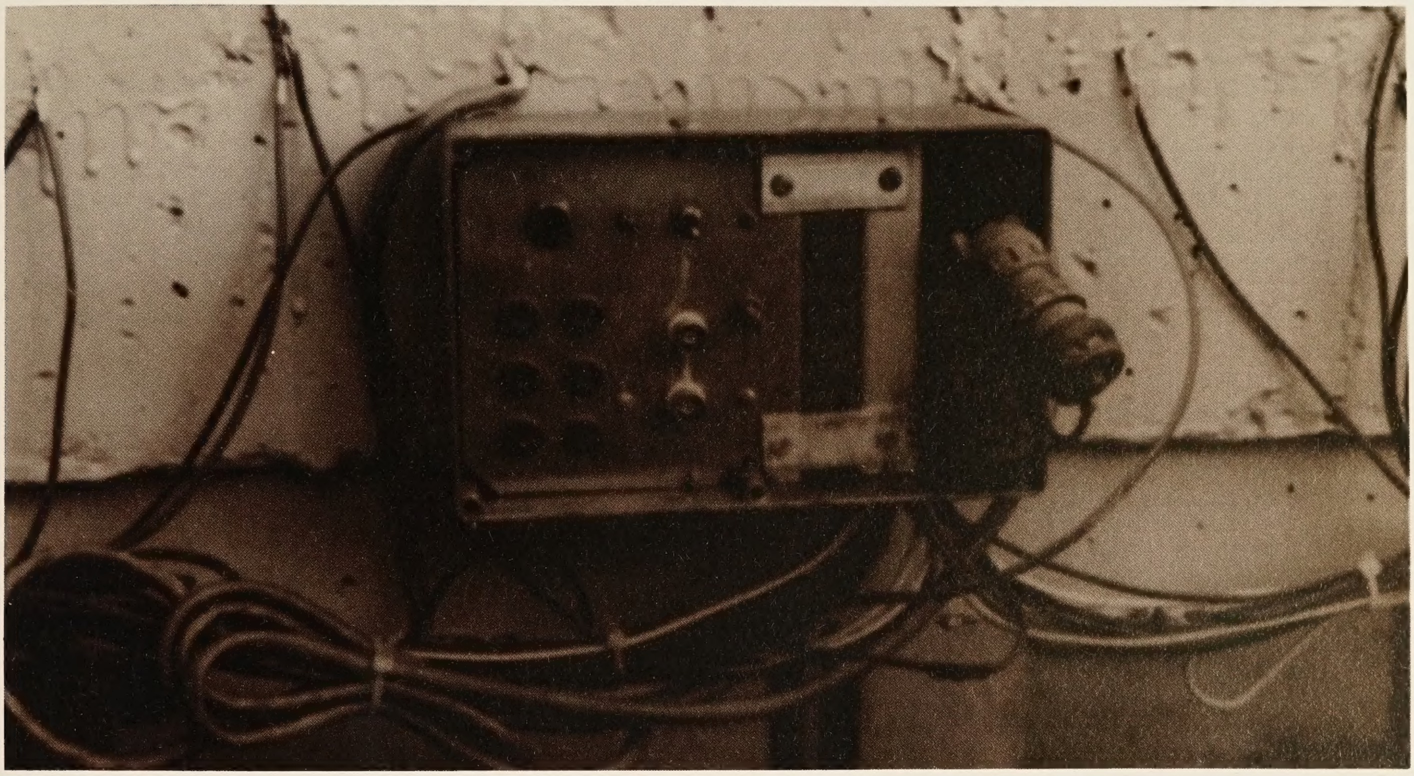
Corrosion of Nonspecification Epoxy-Coated Rebars in Salty Concrete

by
Kenneth C. Clear and Yash Paul Virmani



Introduction

Deterioration of concrete bridge decks, structural members, and substructures is a widespread problem caused primarily by the intrusion of chloride ions from the use of deicers or the splash of saltwater in coastal areas. These chloride ions facilitate the formation of Fe^{++}, which in combination with moisture and oxygen are responsible for the formation of expansive rust products. The following electrochemical reactions probably are responsible for the continuous corrosion of reinforced steel.

Anodic Reactions:

$$Fe^{\circ} + 2Cl^{-} \rightarrow (Fe^{++} + 2Cl^{-}) + 2e^{-}$$

$$Fe^{++}(2Cl^{-}) + 2H_2O \rightarrow Fe(OH)_2 + 2H^{+} + 2Cl^{-}$$

Cathodic Reaction:

$$\tfrac{1}{2}O_2 + H_2O + 2e^{-} \rightarrow 2OH^{-}$$

Small (micro) uniform corrosion present on the same piece of rebar steel embedded in the heterogeneous salt-contaminated concrete is due to the formation of cathodic and anodic sites near each other. In the bridge deck, extensive (macro) corrosion damage is due to the presence of cathodic bottom steel mat in chloride free concrete that discharges large amounts of galvanic current to the anodic sites in the salt-contaminated top steel mat.

Early research has indicated that reinforcing bars coated with select powdered epoxies, using an electrostatic spray process after bar cleaning, perform well in salt-contaminated concrete. (*1–3*)[1] Because such coated rebars are highly resistant to high rates of corrosion, early deterioration of the surrounding concrete, from pressures generated by the expansive corrosion products, is minimized.

As a result, epoxy-coated reinforcing steel is widely used in the construction of new bridge decks in coastal environments and where deicing salts are used. However, more information is needed on the effect of coupling coated and uncoated reinforcing bars in a structure and whether significant further reductions in corrosion rate can be achieved by epoxy coating all the rebar rather than coating only that portion exposed in salty concrete.

To provide such information the Federal Highway Administration (FHWA) initiated an outdoor slab study in 1980 at its McLean, Va., exposure site. This article provides interim data from the continuing study.

[1] Italic numbers in parentheses identify references on page 10.

Research Design and Slab Fabrication

Data on the process of steel corrosion (*2, 3*) indicated that macroscopic corrosion cells (those in which large quantities of cathodic steel, on the same and surrounding reinforcing bars, drive corrosion at the anodic steel) were the primary cause of early bridge deck deterioration and microscopic corrosion cells (those that operate on a single small section of reinforcing steel) were less damaging. These data explain why a few damaged areas on epoxy-coated reinforcing bars do not negate performance for electrically isolated epoxy-coated bars or why when all bars are epoxy coated, a corrosion macrocell of significant magnitude cannot develop because the properly coated portion of the bar cannot function as an oxygen reducing cathode.

Specifically, previous research had shown that a strong and predictable macroscopic corrosion cell developed between top mat uncoated reinforcing steel in chloride contaminated concrete and bottom mat uncoated reinforcing steel in chloride free concrete. The corrosion rate was often controlled by the ability of the bottom mat reinforcing steel to reduce oxygen. A large quantity of bottom mat steel and outdoor exposure were essential in obtaining high corrosion rates. Also, the resistance path between reinforcing bar mats was important.

Thus, a highly accelerated corrosion test with respect to the bridge deck situation would be one in which:

- A relatively large slab was used and stored outdoors.
- Chloride contamination at the level of the top mat reinforcing steel was achieved rapidly.
- A large quantity of bottom mat reinforcing steel (in chloride free concrete) was used.
- The separation distance between rebar mats was reduced to a value less than that typically used on bridge decks.

Of course, even in such a situation a macrocell will not develop unless there is direct electrical contact (metal-to-metal contact) between the rebar mats. In a conventional black steel bridge deck such contact normally is provided by truss bars, tie wires, bar chairs, expansion dams, and/or scuppers. Field testing indicated that in black steel decks such contact was available in virtually every instance.

To research a nonmetallic rebar coating, several bridge decks in Kentucky and Virginia constructed using epoxy-coated reinforcing steel for the top mat and black steel in the bottom mat were evaluated. The data indicated that some of the epoxy-coated reinforcing bars were in electrical contact with the bottom mat steel (table 1). On two decks all the bars tested showed electrical contact, although no contact was found anywhere on four other decks. Partial contact was the most common situation (11 decks). Nonmetallic-coated tie wires and bar chairs seemed to minimize the amount of mat-to-mat coupling but did not always eliminate it. Truss bars, bar ends in contact with expansion dams, and scuppers and associated positioning wires appeared to be the major cause of the electrical contact on the bridge decks.

Therefore, it was decided that a worst case research design would be used for the 1980 FHWA slab study. The criteria mentioned above for a highly accelerated corrosion test would be used and positive electrical contact would be provided between all (100 percent) of the reinforcing bars in each slab.

Each slab was 0.6 m by 1.5 m by 0.15 m (2 ft by 5 ft by 6 in) and contained two mats of reinforcing steel. The top mat reinforcing steel consisted of four 1.3 m (51 in) long bars with 19.1 mm (0.75 in) of clear concrete cover and two 457 mm (18 in) cross bars beneath them. The bottom mat consisted of seven 1.3 m (51 in) long bars positioned 35 mm (1.375 in) from the bottom of the

Table 1.—Electrical resistance between epoxy-coated top and uncoated bottom mat rebars in bridge decks

Project	Number of readings	Resistance, uncoated bottom mat to top mat epoxy bars: Average	Range	Comments
		Ohms	*Ohms*	
Kentucky 1	7	4	0–10	All low
Kentucky 2	15	—	8–infinity	13 at infinity 3 at 8–15 ohms
Kentucky 3	12	Infinity	Infinity	All infinity
Kentucky 4	21	—	0–infinity	14 at infinity 4 at zero 3 at 10–100 ohms
Kentucky 5	36	—	4–infinity	29 at infinity 7 at 4–100 ohms
Kentucky 6	24	—	0–infinity	18 at infinity 6 at zero
Kentucky 7	12	—	10–infinity	10 at infinity 2 at 10 and 15 ohms
Kentucky 8	20	Infinity	Infinity	All infinity
Kentucky 9	20	—	9–infinity	18 at infinity 2 at 9 and 10 ohms
Kentucky 10	20	—	9–infinity	19 at infinity 1 at 9 ohms
Kentucky 11	12	Infinity	Infinity	All infinity
Kentucky 12	36	—	8–infinity	23 at infinity 13 at 5–50 ohms
Kentucky 13	12	Infinity	Infinity	All infinity
Kentucky 14	12	Infinity	9–infinity	10 at infinity 2 at 9 and 10 ohms
Kentucky 15	16	15	9–20	All low
G.W. Parkway 1	20	—	Less than 10–infinity	5 at infinity 15 at less than 10 ohms
G.W. Parkway 2	20	—	Less than 10–infinity	3 at infinity 17 at less than 10 ohms

slab and three 457 mm (18 in) long cross bars beneath them. Figures 1-6 show the slab fabrication, rebar mats, and wiring. All epoxy-coated rebars were No. 6 (19.1 mm [0.75 in] diameter). Black steel of Nos. 4, 5, and 6 (12.7, 16.1, and 19.1 mm [0.50, 0.635, and 0.75 in] diameter, respectively) was used. Table 2 provides specifics on the reinforcing steel in each slab. The separation distance between the mats (longer bars) was 61 mm (2.4 in). The black steel bottom reinforcing steel mats were welded at each bar crossing point. Electrical leads were attached on each epoxy-coated rebar, all top mat black steel rebars, and two of the rebars in each black steel bottom mat in the following manner: The ends of the bars were sandblasted and a 25.4 mm (1 in) wide area on one side of the bar was flattened using a metal lathe. A 6.4 mm (0.25 in) diameter hole was drilled through the bar. A 6.4 mm (0.25 in) nut and bolt and washers, tightened to 27 J (20 ft-lb), were used to attach the lead wire (with closed ring crimp lug with No. 8 hole) to the bar. The attachment area was then well coated with epoxy. All lead wires were No. 12 stranded, tinned copper with Teflon insulation.

The concrete used in each slab had a water-cement ratio of 0.53, cement content of 390 kg/m^3 (658 lb/yd^3), air content of 7 $\pm$1.5 percent, and 44 percent sand by volume of total aggregate. The fine aggregate was White Marsh sand (specific gravity of 2.64 and fineness modulus of 2.6). The coarse aggregate was Riverton Limestone (specific gravity of 2.77 and 19.1 mm [0.75 in] maximum size) graded to the midpoint of the AASHTO M43 size No. 67 specification. All coarse aggregate, screened to four separate sizes, was batched separately to insure gradation control.

The concrete was mixed in a 0.3 m^3 (11 ft^3) rotary drum mixer in 0.25 m^3 (9 ft^3) batches. The concrete in each slab was placed in two lifts 1 to 3 days apart. The lower lift 89 mm (3.5 in) was chloride free, but the concrete in the top lift of each slab contained 8.9 kg Cl$^-$/m^3 (15 lb Cl$^-$/yd^3), added by placing sodium chloride in a portion of the mix water. Figures 1 and 2 show the placement sequence. The lower lift

Table 2.—Research design and slab fabrication data, epoxy-coated rebars and controls

Slab	Lift	Date made	Mix temperature	Cl$^-$ [1]	Unit weight	Air	Reinforcing mat
			Degrees Fahrenheit	*lb/yd^3*	*lb/ft^3*	*Percent*	
201	Top	5/9/80	68	15	139	6.8	Black #4 trans; #5 long[2]
	Bottom	5/6/80	79	0	137	6.9	Black #4 trans; #5 long
202	Top	5/12/80	82	15	—	7.0	Black #4 trans; #5 long
	Bottom	5/10/80	77	0	142	5.6	Black #4 trans; #5 long
234	Top	9/28/80	79	15	141	7.8	Black #6 trans; #6 long
	Bottom	9/27/80	77	0	133	9.0	Black #4 trans; #5 long
203	Top	5/9/80	68	15	139	6.8	Epoxy #6 trans; #6 long
	Bottom	5/6/80	79	0	137	6.9	Black #4 trans; #5 long
204	Top	5/12/80	79	15	142	6.4	Epoxy #6 trans; #6 long
	Bottom	5/10/80	77	0	142	5.7	Black #4 trans; #5 long
205	Top	5/12/80	79	15	142	6.4	Epoxy #6 trans; #6 long
	Bottom	5/10/80	77	0	142	5.7	Epoxy #6 trans; #6 long
206	Top	5/12/80	82	15	—	7.0	Epoxy #6 trans; #6 long
	Bottom	5/10/80	77	0	142	5.6	Epoxy #6 trans; #6 long
207	Top	5/15/80	72	15	139	7.3	Epoxy #6 bare ¼″ x ½″ every 6″
	Bottom	5/13/80	84	0	141	6.0	Black #4 trans; #5 long
208	Top	5/16/80	69	15	145	6.1	Epoxy #6 bare ¼″ x ½″ every 6″
	Bottom	5/13/80	89	0	142	5.5	Black #4 trans
209	Top	5/15/80	72	15	139	7.3	Epoxy #6 bare ¼″ x ½″ every 18″
	Bottom	5/13/80	84	0	141	6.0	Black #4 trans; #5 long
210	Top	5/16/80	69	15	145	6.1	Epoxy #6 bare ¼″ x ½″ every 18″
	Bottom	5/13/80	89	0	142	5.5	Black #4 trans; #5 long
235	Top	9/28/80	79	15	141	7.8	Epoxy #6 bare ¼″ x ½″ every 6″
	Bottom	9/27/80	77	0	133	9.8	Epoxy #6 bare ¼″ x ½″ every 6″

[1]Chloride contents shown are theoretical, based on the amount of NaCl added to the concrete mixing water.
[2]trans=transverse; long=longitudinal.

°C=(°F-32)/1.8
1 lb/yd^3=0.593 kg/m^3
1 lb/ft^3=16.02 kg/m^3
1 in=25.4 mm

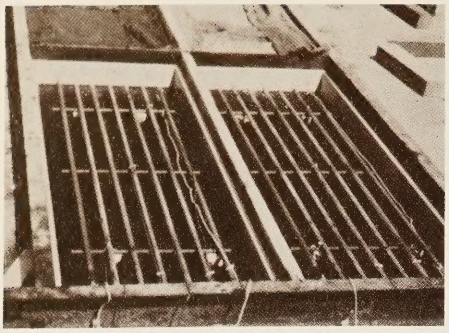


Figure 1.—Bottom rebar mats.

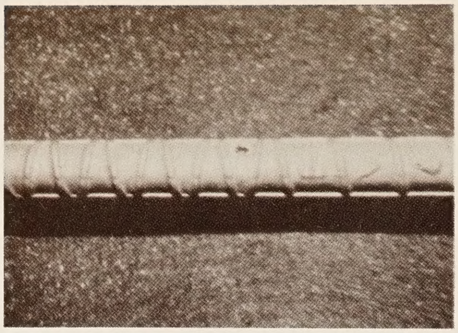


Figure 3.—Bare area on epoxy-coated rebar.

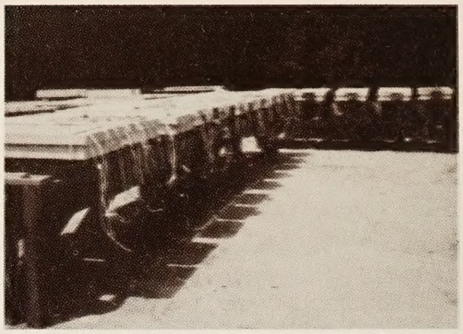


Figure 5.—Slabs at outdoor test yard before wiring.

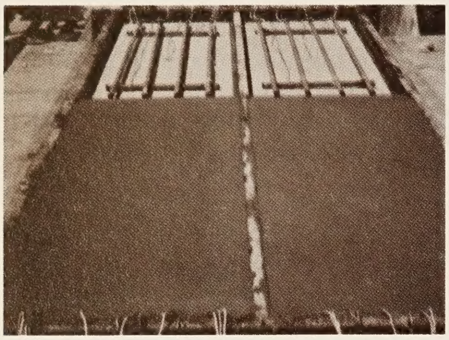


Figure 2.—Top rebar mats and completed slabs.

Figure 4.—Completed slabs.

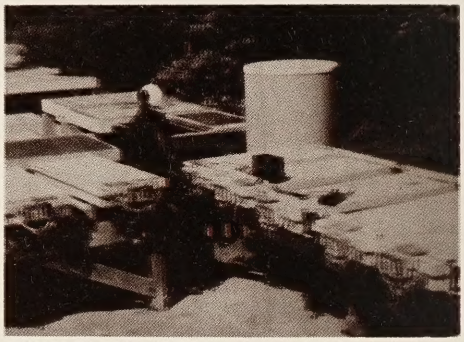


Figure 6.—Slabs at outdoor test yard after wiring.

was cured with wet burlap and then wire brushed before top lift placement. The top lift was cured for 14 days after placement using wet burlap and polyethylene and then mounted on 0.9 m (3 ft) posts at the FHWA outdoor exposure site. A tendency for the concrete to crack between lifts became apparent after the first slabs were removed from curing. To minimize this, angle iron clamps were placed on each end of each slab. The clamps were electrically isolated from the slabs using 6.4 mm (0.25 in) thick plexiglass strips. Electrical potential, resistance, and corrosion current measurements on slabs with and without the clamps confirmed that the clamps had no effect on the corrosion evaluation. Table 2 provides specifics on each slab and the concrete. All slabs were fabricated between May 6 and September 28, 1980. Nine thermocouples (three at the top mat level, three at the slab middepth, and three at the bottom mat level) were placed in each slab to accurately measure the average slab temperature. All lead wires were brought outside the concrete to facilitate corrosion measurements.

The variables under study in the epoxy-coated rebar research effort all involve the reinforcing steel and the presence or absence of an epoxy coating thereon. The epoxy-coated reinforcing steel was coated in 1977 for use in another study and had been stored outdoors for over 2 years. Holiday detection was used to identify those bars with more than 25 holidays per 0.3 m (1 ft) and only these bars were used in this study. Also, the epoxy-coated rebar did not pass the bend test. Upon being subjected to this test, it was easy to peel the epoxy coating from the rebar. This test is used to detect poor surface preparation that will result in a poor bond between the epoxy coating and the steel. Consequently, the epoxy-coated rebar used in this study did not meet AASHTO or ASTM specifications for the finished product. (*4, 5*)

A small amount of shipping and storage chipping was present on the coated rebar (estimated at less than 0.05 percent of the rebar surface area and less than one visible chip per 0.3 m [1 ft]). All bars were cut to size from the 6.1 m (20 ft) lengths, lead wires were attached, and all ends were coated with liquid epoxy. The bars then were allocated using a random numbers table for use in the various slabs. Coating thickness measurements, made using a thumbwheel gage calibrated on plates, averaged 0.33 mm (12.8 mils) with a range of 0.28 to 0.37 mm (11 to 14.6 mils). This thickness, higher than normal, was requested by FHWA for an earlier bond study.

Damaged areas were created on the epoxy-coated bars for use in slabs 207–210 and 235. Field data were used to define a realistic amount of damage to produce. Preliminary reports indicated that the actual coating damage occurring during routine field use was far less than the 2 percent damaged area allowable by most specifications. FHWA, therefore, requested that the States of Kentucky and Iowa survey epoxy-coated rebar installations, immediately before concrete placement, to define the actual damaged bar areas. Kentucky Department of

Transportation personnel surveyed 16 bridge decks after the steel had been placed. Visual standards (rebars on which bare areas of 0.01, 0.1, and 1.0 percent of surface area had been placed) were used to define the damage on at least one 2.3 m^2 (25 ft^2) representative area of the deck steel of each bridge. Several areas were studied on some bridges. All of these Kentucky bridges were built using epoxy-coated reinforcing steel in the top mat only and nonmetallic-coated chairs and tie wires. On 12 decks the average damaged area was between 0 and 0.01 percent. The average damaged area on three other decks was between 0.01 and 0.04 percent; although, on a single deck a damaged area of 0.4 percent was found. In the latter case, the inspector indicated that he purposely chose that sampling location because it was the worst area on the deck. Other areas exhibited less damage.

The Iowa study involved the indepth analyses of 30 No. 11 bars and 6 No. 5 bars randomly chosen from three groups at a jobsite immediately before the bars were installed as main column bars and hoop ties. The bars were coated in North Carolina and fabricated as required and shipped by truck to Ames, Iowa. This jobsite was chosen because it was near the FHWA division office. Preliminary examination indicated the column bars were more badly damaged than bridge deck reinforcing steel. In this study, the percentage of damaged area was defined in each 0.3 m (1 ft) interval of each bar. The size of each damaged area was estimated by comparison with an area representation card. The card contained 18 shaded squares or rectangles varying in area from 2.5 mm^2 to 60.6 mm^2 (0.0039 in^2 to 0.094 in^2).

Most of the damage on these bars was found on the ridges near the ends of the bar. The maximum amount of defective area in any 0.3 m (1 ft) length was 1.08 percent for the No. 5 bars and 0.88 percent for the No. 11 bars. However, overall averages for the bars were 0.22 percent for the No. 5 hoop bars and 0.065 percent for the No. 11 bars.

The above detailed studies confirm the earlier belief that damaged areas on epoxy-coated reinforcing steel generally are less than 2 percent of the bar surface area. Therefore, it was decided to use damaged areas of 0.24 and 0.86 percent on the bars for the slabs in this study. A 6.4 mm by 12.7 mm (0.25 in by 0.50 in) damaged area (80 mm^2 [0.124 in^2]) was chosen. A 0.24 percent damaged area involved removing the coating from an area that size every 457 mm (18 in) along the No. 6 bars. A 0.86 percent bare area involved coating removal every 152 mm (6 in) along the bars (10 and 36 individual damaged areas per top mat, respectively). The damaged points on each bar were placed alternately on each side of the bar along its length. Figure 3 shows a typical damaged area on the coated reinforcing steel.

Further, to insure that a highly corrosive environment existed, all slabs were continuously ponded with 3 percent sodium chloride solution for 46 days in the fall of 1980. The length of ponding was defined as the time required for corrosion to be induced on a control slab that initially was chloride free. After ponding, the dams were removed; since then, the slabs have been subject to natural weathering only.

Corrosion Testing

Previous research had shown that if the only mat-to-mat electrical contact was exterior to the slab, a direct measure of the corrosion current flowing between the mats could be obtained. (*6*) Such a measurement is very valuable because it provides direct evidence of corrosion and its magnitude and facilitates the calculation of a variety of important parameters such as the oxygen consumption rate at the cathode. Also, by monitoring corrosion current versus time, a valid indication of the iron consumed by the action of the macroscopic corrosion cell can be obtained.

An instrumentation interface box was attached to each slab, and all lead wires were attached to the box to allow the following:

(a) A switch couples or uncouples all the rebars in the top mat to/from all the rebars in the bottom mat.

(b) The corrosion current flowing between the two mats can be measured directly as the voltage drop across at 1.0 ohm precision resistor (switch coupled).

(c) The driving voltage of the corrosion cell can be measured (at switch instant OFF).

(d) The electrical potential between the top mat rebar and a half-cell placed at various positions on the slab surface can be measured (switch OFF).

(e) The electrical potential between the bottom mat rebar and a half-cell placed at various positions on the bottom surface of the slab can be measured (switch OFF).

(f) The electrical resistance between rebar mats can be measured using a 1,000 cycle AC meter (switch OFF).

(g) The temperatures indicated by the nine thermocouples within the slab can be recorded (switch ON or OFF).

Once a slab is under test, the mat couple switch is ON except when certain measurements are made. The typical sequence is to make measurements (b) and (g) with the switch ON, measurement (c) immediately upon uncoupling, and then measurements (d), (e), and (f) with the switch OFF. The mats are then recoupled. It is essential that the corrosion current measurement be made first (before uncoupling) because depolarization of the corrosion cell is rapid after uncoupling. Testing showed that typically 1 to 2 days in the mats-coupled mode were required for the corrosion cell to stabilize (polarize to a steady state condition) initially or after several days in the uncoupled mode. Shorter stabilization times (1 hour) were required for short uncoupled periods, such as that needed to obtain the above measurements.

The corrosion testing of slabs 201–210 (epoxy coated and controls) was initiated on August 7, 1980, by coupling the reinforcing steel mats after a set of initial uncoupled data had been obtained. A similar procedure was used to initiate the testing of slabs 234 and 235 (epoxy bar and control) on October 28, 1980. The reinforcing steel then remained coupled throughout the test except when data were being collected.

Rate of Corrosion Findings

Table 3 summarizes the rate of corrosion data obtained on the slabs. Included are data on the following:

• Average macrocell corrosion current—This is a direct measure of the electrons released by the corrosion process and flowing to the bottom rebar mat for oxygen reduction. As noted above, concrete temperature has a significant effect on corrosion current (rate). This effect primarily is from the effect of temperature on concrete resistivity. Also, it has been shown that the corrosion current measured at any given field temperature can be adjusted to another temperature to compensate for differing concrete resistivities using the formula (*2*):

$$i_1 = \frac{i_2}{e^{2883\left(\frac{1}{T_1} - \frac{1}{T_2}\right)}}$$

Where,
i_1 = Corrosion current at temperature T_1
i_2 = Corrosion current measured at temperature T_2
T_1 = Temperature (in degrees Kelvin) that one desires to know the corrosion current
T_2 = Average temperature (in degrees Kelvin) of the concrete between the macroanode and macrocathode.

A temperature of 21° C (70° F) was chosen for T_1 in these studies, and all measured currents were adjusted to a concrete resistivity corresponding to that temperature.

Two averages are presented: An arithmetic average obtained by adding all data and dividing by the number of readings and a weighted average that considers the variable time intervals between data points. The latter is considered the better indicator.

• Average macrocell driving voltage—This is the polarized driving voltage of the corrosion cell measured in the instant-OFF mode (an instant after uncoupling the rebar mats). If no corrosion macrocell developed, the driving voltage would be zero. At constant corrosion circuit resistance, the higher the driving voltage, the higher the rate of corrosion.

• Mat-to-mat AC electrical resistance—This measurement, made using a 1,000 cycle AC signal after uncoupling the mats and measuring the electrical potentials, indicates concrete resistivity when black steel, or rebar coated with a metallic material that adds little circuit resistance, is used. The mats are recoupled immediately after making this measurement. Tests in solutions of known resistivity were used to define a resistance-to-resistivity conversion factor of 735 for the black steel slabs with all No. 4 and No. 5 bars and 706 for the black steel slab with No. 6 bars. By multiplying these conversion factors by the resistance measured, the approximate concrete resistivity is defined. However, such an approach cannot be used for the slabs with epoxy-coated rebars. In general, the higher the concrete resistivity, the lower the corrosion current. For corrosion currents, field measurements are adjusted to 21° C (70° F) using an experimentally defined equation.

• Metal consumed, 21° C (70° F)—This is the amount of metal that would have been consumed during the test period if each concrete resistivity had constantly been at its 21° C (70° F) adjusted value. It is well known that each 1.0 amp-hour of corrosion current consumes 1.04 g (0.04 oz) of iron. Total amp-hour of current passed is calculated by multiplying the average corrosion current for each two successive readings by the hours between readings and accumulating a total.

The rate of corrosion data indicate that even nonspecification epoxy-coated reinforcing steel is quite

Table 3.—Corrosion rate data

Ponded slab	Variable	Average driving voltage	Average 70° F corrosion current	Weighted average 70° F corrosion current	Average 70° F mat-to-mat resistance	70° F metal consumed per year
		mV	*µA*	*µA*	*Ohms*	*Grams*
201[1]	Black steel	141	2,958	3,102	42.0	28.30
202	Black steel	87	6,089	6,478	11.0	59.00
234	Black steel	92	6,521	6,663	10.5	60.70
205	Epoxy, both mats	18	106	135	100.5	1.23
206	Epoxy, both mats	22	115	140	90.3	1.28
235	Epoxy, both mats with 0.86% damage	22	122	139	56.6	1.26
203	Epoxy, top only	25	301	362	53.0	3.29
204	Epoxy, top only	43	722	876	43.5	7.98
207[1]	Epoxy, top only and 0.86% bare	75	516	534	100.5	4.86
208	Same as 207	29	477	518	36.3	4.71
209	Epoxy, top only and 0.24% bare	28	273	311	54.8	2.84
210	Same as 209	36	334	381	63.7	3.47

[1]Debonded between lifts so not included in average calculations in the text.

°C=(°F-32)/1.8

effective in reducing corrosion, although some corrosion did occur in all the highly chloride bearing concrete. Little differences were found between bars with holidays only and those with holidays and visible bare areas. Therefore, the data have been grouped into three primary areas (black steel, top mat only epoxy coated, and both mats epoxy coated) and are summarized in table 4.

Table 4.—Average corrosion current 70° F for black and epoxy-coated slabs

Variable	Weighed average 70° F corrosion current	Ratio to black steel
	Amps	
Black (uncoated), both mats	6,571	
Black (uncoated), both mats[1]	5,650	1 to 1
Epoxy coated, top mat only	490	11.5 to 1
Epoxy coated, both mats	138	41 to 1

[1]By interpolation for the same amounts of chloride as present in epoxy-coated slabs. °C=(°F-32) 1.8

These data show that corrosion can be reduced greatly by using even poor quality epoxy-coated reinforcing steel and a worst case situation in which all bars are electrically coupled. When only one mat was coated, it would take an average of 11.5 years to consume the same quantity of iron that was consumed in 1 year on uncoated steel. If all rebar is epoxy coated, it would take 41 years to consume the same amount of iron.

Means of Protection

The electrical half-cell potential, macrocell driving voltage data, and electrical resistance data reveal how the protective systems are functioning (tables 3 and 5).

Table 5.—Average electrical half-cell potentials

Slab	Variable	Average electrical potentials Top mat	Bottom mat	Average potential difference
		mV CSE	*mV CSE*	*mV*
201[1]	Black steel	-503	-216	-287
202	Black steel	-479	-243	-236
234	Black steel	-532	-307	-225
203	Epoxy, top only	-251	-140	-111
204	Epoxy, top only	-297	-154	-143
207[1]	Epoxy, top only and 0.86% bare	-364	-173	-191
208	Same as 207	-263	-140	-123
209	Epoxy, top only and 0.24% bare	-269	-160	-109
210	Same as 209	-248	-133	-115
205	Epoxy, both mats	-342	-244	-98
206	Epoxy, both mats	-383	-263	-120
235	Epoxy, both mats and 0.86% bare	-465	-316	-149

[1]Debonded between lifts so not included in average calculations in the text.

The mat-to-mat resistance data indicate that epoxy-coated reinforcing steel owes its success largely to the increased macrocorrosion cell resistance path (average mat-to-mat resistance for uncracked slabs equals 82.5 ohms and 50.3 ohms for both mats coated and one mat coated, respectively, versus 10.8 ohms for all black steel). Further, the macrocell driving voltages and half-cell potential differences are far lower for all epoxy bar slabs than for the black steel controls. Top mat potentials in the one mat epoxy-coated situation did not become highly negative because sufficient oxygen was available at the top mat rebar level to support microcell action at holidays and bare areas without microcathode polarization. (Had this not been the case, top mat potentials would have been more negative because of higher macrocell corrosion rates.)

From the above, better performance could be expected when both mats are coated because of the higher mat-to-mat resistance and lower macrocell driving voltage (both mats coated average equals 21 mV and top mat only average equals 32 mV). A major difference in electrical half-cell potentials (polarized) exists, however, for these two situations: Both top and bottom mat polarized potentials are much more negative when both mats are coated than when only the top mat is coated. This indicates that macrocathode polarization is occurring when both mats are coated undoubtedly because of the limited bare steel area on the bottom mat for oxygen reduction. Potential data (both mats coated) obtained before mat coupling indicate that the unpolarized bottom mat potentials were substantially more positive than the polarized values.

Thus, control of the corrosion rate for one mat epoxy coated is best characterized as resistance control with the absence of top mat microcathode polarization; both mats coated involves both resistance effects and macrocathodic polarization.

As noted earlier, several slabs debonded between concrete lifts (between the mats) during curing. Examination of the data, in comparison with that for bonded slabs, shows that resistivity effects are playing an important role in the debonded slabs and that cathodic polarization generally is reduced, resulting in higher macrocell driving voltages and mat-to-mat electrical potential differences. Therefore, the debonding between lifts in the black steel slabs caused a shift in the balance of corrosion rate control from predominately cathodic control to predominately resistance control.

Visual Examination for Confirmation

In August 1981, selected slabs were demolished to visually examine the reinforcing steel (figs. 7–12). Chloride contents at various depths within the concrete were defined using rotary hammer sampling and the procedure defined in AASHTO T–260. A discussion on the findings for each slab is given below.

Slab 201—Widespread rebar corrosion on all bars, with (as usual) some uncorroded (cathodic) areas. Corrosion-induced cracking throughout. Metal loss at some spots is significant.

Slab 204—Little corrosion found; hairline cracking is not corrosion related. Minor surface scaling (due to freeze thaw action immediately after ponding). Several spots of rusting seen at holidays.

Slab 205—Very little corrosion on bars or corrosion product in concrete. No corrosion-induced cracking.

Slab 209—Little rebar corrosion found. A few spots were identified. No corrosion-induced cracking. Minor surface scaling.

In general, the slab demolition confirmed the findings of the rate of corrosion studies and validated the test technique. The epoxy-coated rebar slabs had much less steel corrosion than the control slab. Corrosion in the black steel slab had induced widespread cracking; none was found in the epoxy-coated rebar slabs.

Although the epoxy-coated rebar performed from 11.5 to 41 times better than the black steel bars, there was light rusting under the epoxy coating at certain locations. Because the bars had not been properly cleaned before coating, the bond between the coating and the bar was unsatisfactory. This would allow corrosion to advance under the coating from any point of entry such as a holiday or where the coating had been removed to simulate a damaged area.

Figure 7.—Slab 201: Black steel, both mats, 8.9 kg Cl^-/m^3 (15 lb Cl^-/yd^3).

Figure 8.—Slab 201: Demolished, visible rust on iron rebar mat.

Figure 9.—Slab 201: Demolished, concrete with rebar rust imprint.

Figure 10.—Slab 204: Epoxy coated with holidays, top mat only, 8.8 kg Cl^-/m^3 (14.9 lb Cl^-/yd^3).

April 1982 slab condition

The condition of each of the remaining slabs, as of April 2, 1982, is shown in figures 13–20. The black steel control slabs with 8.9 kg Cl^-/m^3 (15 lb Cl^-/yd^3) are badly corroded; none of the epoxy bar slabs shows corrosion-induced distress. In general, the slab condition in April 1982 indicates that the previous findings of the rate of corrosion studies remain valid.

Conclusions

Concrete structures constructed in salt environments using epoxy-coated reinforcing steel should be many, many times more resistant to corrosion-induced concrete damage than those constructed with uncoated rebar. This is true even though the epoxy-coated rebars did not meet either the holiday requirement (pinholes not discernible by the unaided eye) or the bend test as specified by AASHTO M 284 or ASTM D 3963. However, this does not mean requirements should be relaxed but rather it reinforces the validity of these specification requirements. Over the very long term the development of corrosion

Figure 11.—Slab 204: Demolished, no visible rust on epoxy-coated rebars.

Figure 12.—Slab 204: Demolished, concrete with rebar imprint but no rust.

beneath the epoxy coating might become detrimental. These requirements initially were placed in the quality control portion of the specifications to obtain the same quality of product that was evaluated in the prequalification portion of the specifications for resistance to applied voltage (accelerated corrosion test).

The fusion-bonded epoxy coating functions well mainly because of the high electrical resistance of the coating and by greatly reducing the steel surface area available for cathodic oxygen reduction. This greatly reduces the total metal consumed, the major cause of concrete disruption, even though the corrosion current density at local coating breaks may be quite high. Such localized high current density areas are not of great concern because rebar metal loss is not normally a critical factor. For example, studies have indicated that corrosion-induced concrete cracking occurred

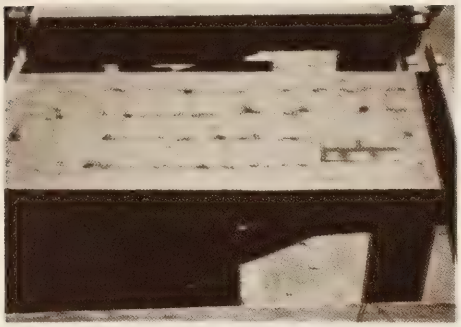


Figure 13.—Slab 202: Black steel, both mats, 9 kg Cl^-/m^3 (15.2 lb Cl^-/yd^3), April 1982.

Figure 14.—Slab 234: Black steel, both mats, 11.3 kg Cl^-/m^3 (19 lb Cl^-/yd^3), April 1982.

Figure 15.—Slab 206: Epoxy coated with holidays, both mats, 8.4 kg Cl^-/m^3 (14.2 lb Cl^-/yd^3), April 1982.

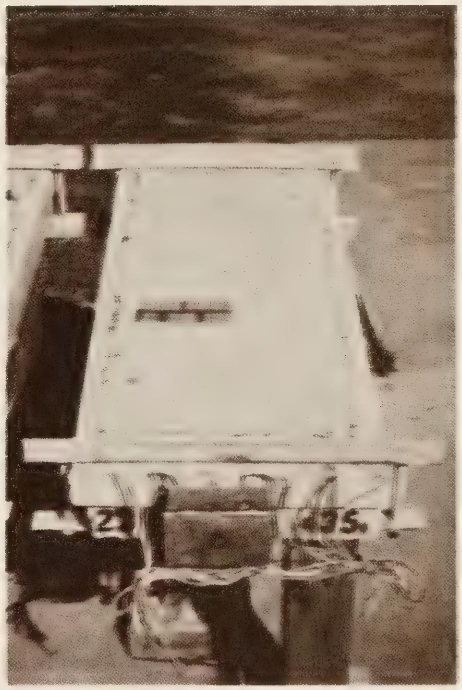


Figure 16.—Slab 235: Epoxy coated with holidays, 0.86 bare area, both mats, 11.6 kg Cl^-/m^3 (19.6 lb Cl^-/yd^3), April 1982.

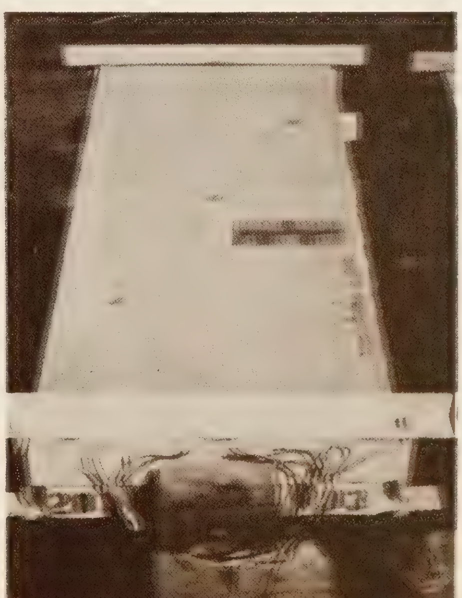


Figure 17.—Slab 203: Epoxy coated with holidays, top mat only, 8.6 kg Cl^-/m^3 (14.5 lb Cl^-/yd^3), April 1982.

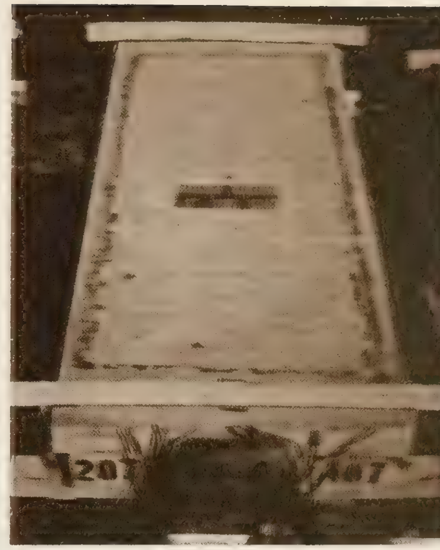


Figure 18.—Slab 203: Epoxy coated with holidays, top mat only, 8.6 kg Cl^-/m^3 (14.5 lb Cl^-/yd^3), April 1982.

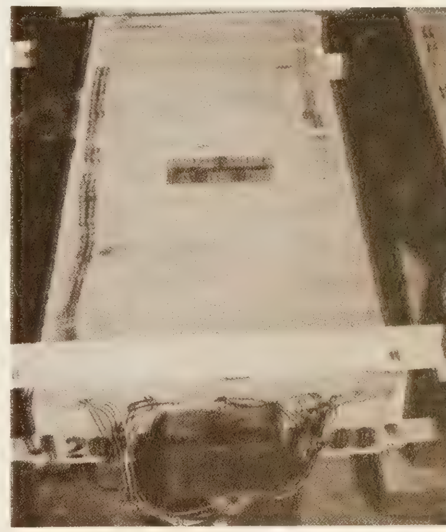


Figure 19.—Slab 208: Epoxy coated with holidays, 0.86 percent bare area, top mat only, 9.3 kg Cl^-/m^3 (15.6 lb Cl^-/yd^3), April 1982.

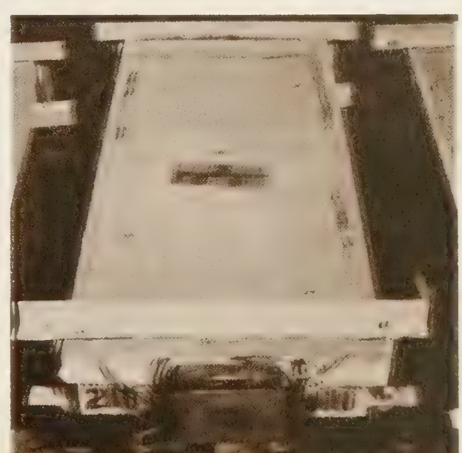


Figure 20.—Slab 210: Epoxy coated with holidays, 0.24 percent bare area, top mat only, 8.7 kg Cl^-/m^3 (14.7 lb Cl^-/yd^3), April 1982.

after only 0.5 to 1 percent of the bar steel was consumed; however, reinforcing steel production specifications typically allow a production variation of at least 6 weight percent. (*2*) Further, no deep pitting was found at bare areas when select slabs were demolished.

The best situation is one in which all the bars are epoxy coated or the coated bars are electrically isolated from other metal in the structure. However, even when nonspecification epoxy-coated bars in salty concrete were all electrically coupled to large amounts of uncoated steel in salt free concrete, total metal consumed was less than one-tenth of that consumed when using all uncoated bars.

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(*4*) The American Association of State Highway and Transportation Officials, Standard Specification for Epoxy-Coated Reinforcing Bars, AASHTO Designation M 284-81, Washington, D.C., 1981.

(*5*) American Society for Testing and Materials, Standard Specification for Epoxy-Coated Reinforcing Steel, Designation D 3963-81, ASTM, Philadelphia, Pa., 1981.

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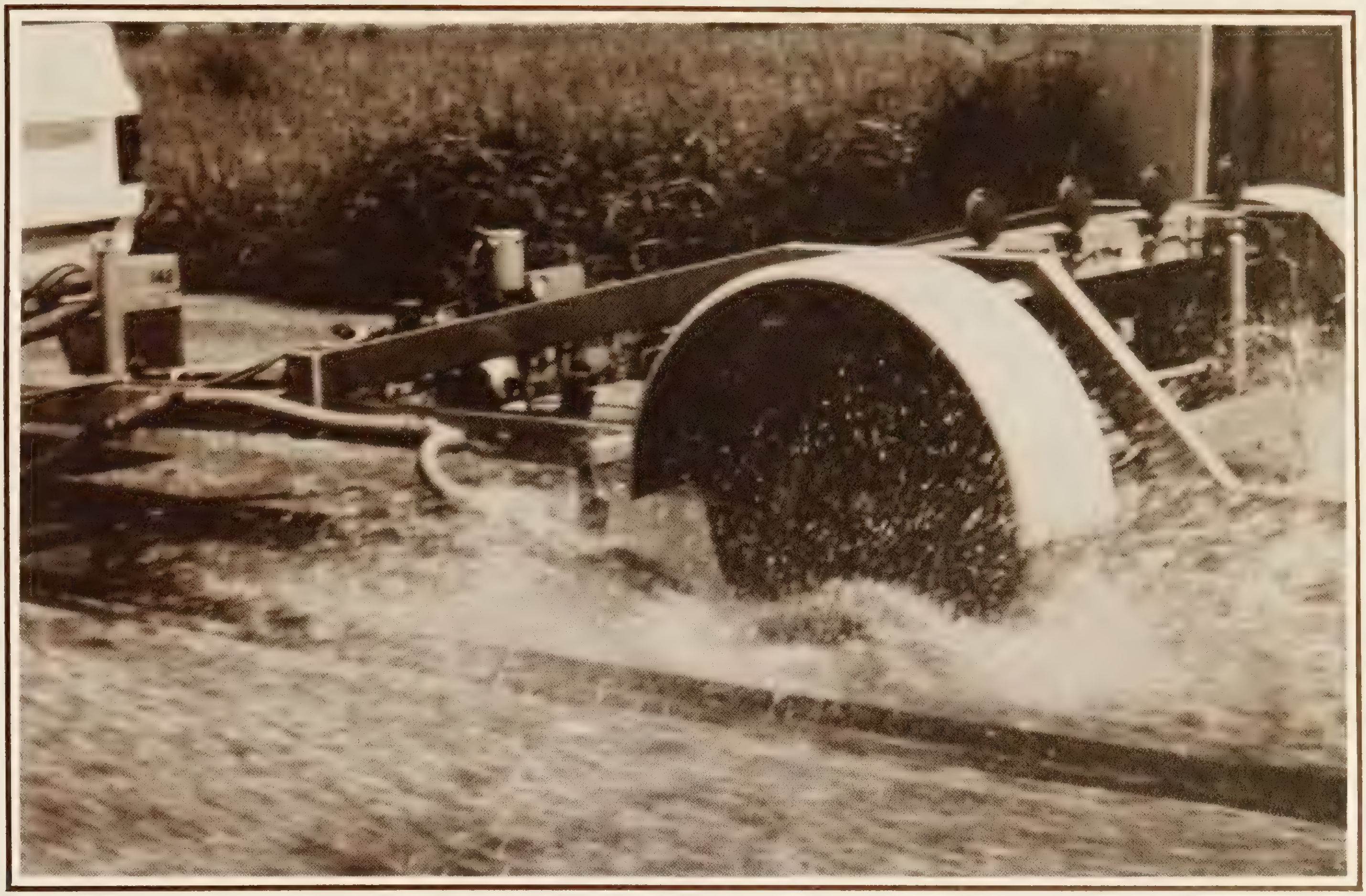
[2]Report with PB number is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.



Hydroplaning—How to Reduce It[1]

by
Glenn G. Balmer and Bob M. Gallaway

This article discusses controls, such as vehicle operation restrictions and pavement design, to decrease hydroplaning, which is affected by pavement cross slope, texture, rut depth, surface drainage, drainage path length, precipitation intensity and duration, tire inflation and tread pattern depth, and vehicle traveling speed. The recommended vehicle and pavement design controls are based on analyses of simulation, laboratory, and full-scale tests. The results are applicable to vehicle operation, tire construction, and highway pavement design, construction, and rehabilitation.

Introduction

Automotive hydroplaning occurs when water on the roadway separates the tires of a moving vehicle from the road surface. Partial hydroplaning, which also is quite hazardous, is prevalent during vehicle operations whenever there is an appreciable amount of water on the road.

The likelihood of full dynamic hydroplaning is small, because the probability of combined occurrence of factors that cause it is low. For example, high intensity rainfalls are rare, and it is unlikely that many vehicles will travel at high speeds during such rainfalls. However, full hydroplaning is so hazardous when it does occur that pavement design and vehicle operating controls to reduce it are warranted.

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[1]This article is a condensation of an American Society for Testing and Materials paper. See reference 1.

Controls to Reduce Hydroplaning

Analyses of experimental data show that hydroplaning can be decreased by observing the nature, intensity, duration, and intermittence of precipitation and by applying vehicle and pavement controls. (*1–4*)[2]

Vehicle controls include the following:

- Decreasing travel speed on wet pavements.
- Using tires with adequate tread pattern depth and normal tread width.
- Inflating tires to the maximum recommended pressures.

Pavement design, construction, and maintenance controls include the following:

- Providing adequate pavement cross slope.
- Constructing and maintaining pavements with gritty, coarse surface textures or finishes.
- Minimizing water film thickness on pavement surfaces by effective drainage.
- Minimizing pavement drainage path length by roadway design and construction.
- Providing adequate water removal facilities for sag-vertical curves.
- Reducing ponding of water in pavement ruts by proper maintenance.

Precipitation and Its Influence

A comprehensive analysis has been developed to evaluate rainfall intensities in several States. (*2*, *5*) In Illinois, for example, rainfall intensities greater than 150 mm/h (6 in/h) occur less than 2 minutes in 18,521 minutes of rainfall. Precipitation rates greater than 100 mm/h (4 in/h) occur less than 12 minutes, and those greater than 50 mm/h (2 in/h) occur less than 107 minutes.

These results illustrate that high intensity rainfalls are rare and of short duration. When intense rainfalls do occur, most motorists will decrease their travel speed accordingly or stop. Of course, water will collect in depressions in the roadway even from low intensity rainfalls, and hydroplaning can occur from ponded water.

[2]Italic numbers in parentheses identify references on page 19.

Skid Resistance and Hydroplaning Tests

Many tests have been conducted on pavements, on skid pads, and in water troughs on underlying pavement surfaces to study skid resistance and hydroplaning. (*2–4*) Skid trailers, hydroplaning trailers, or other research equipment measured longitudinal and normal tire forces during locked-wheel braking, torque on the trailer wheel axle during braking, slip resistance during cornering, or wheel spin down (deceleration of the rotational wheel speed from hydrodynamic forces) during travel. The test data were analyzed and interpreted as skid numbers, slip numbers, or hydroplaning.

The ratio of the longitudinal force to the wheel load at the tire-pavement interface yields the coefficient of friction between the pavement surface and the test tire when a friction test is conducted in accordance with American Society for Testing and Materials (ASTM) E 274. The coefficient multiplied by 100 is defined as the skid number, SN.[3]

Specification ASTM E 274 requires the quantity of water (internal source) applied at a test speed of 64 km/h (40 mph) to be 0.60 L ± 10 percent/min·mm (4.0 gal ± 10 percent/min·in) of wetted width. With the internal watering system, the same quantity of water is dispersed on different pavements with the skid equipment operating at a particular speed regardless of the pavement texture depth. Therefore, the water film thickness will vary for the different pavement textures.

With an external watering system, however, the pavement texture is considered. A positive water depth is measured from the top of the asperities to the water surface, and the distance from the top of the exposed asperities to the water surface below is a negative water measurement.

The water film thickness was greater for most hydroplaning tests than for skid tests. Pavement type and texture; tire construction, tread, and inflation; water depth; and vehicle speed were varied during the test series. A 10 percent wheel spin down, a very low longitudinal force at the tire-pavement interface during testing, or a low skid number indicated hydroplaning for the thicker water films.

Because this article is concerned principally with dynamic hydroplaning, only a few examples from the test series will be discussed; other test results are given in the references.

[3]The term skid number in this article refers to test values in general, even for exceptions to the strict requirements of the ASTM E 274 standard test that specifies a testing speed of 64 km/h (40 mph), a water film thickness of 0.5 mm (0.02 in), and the use of the standard test tire. Except for terminology, this test method is the same as American Association of State Highway and Transportation Officials (AASHTO) T 242.

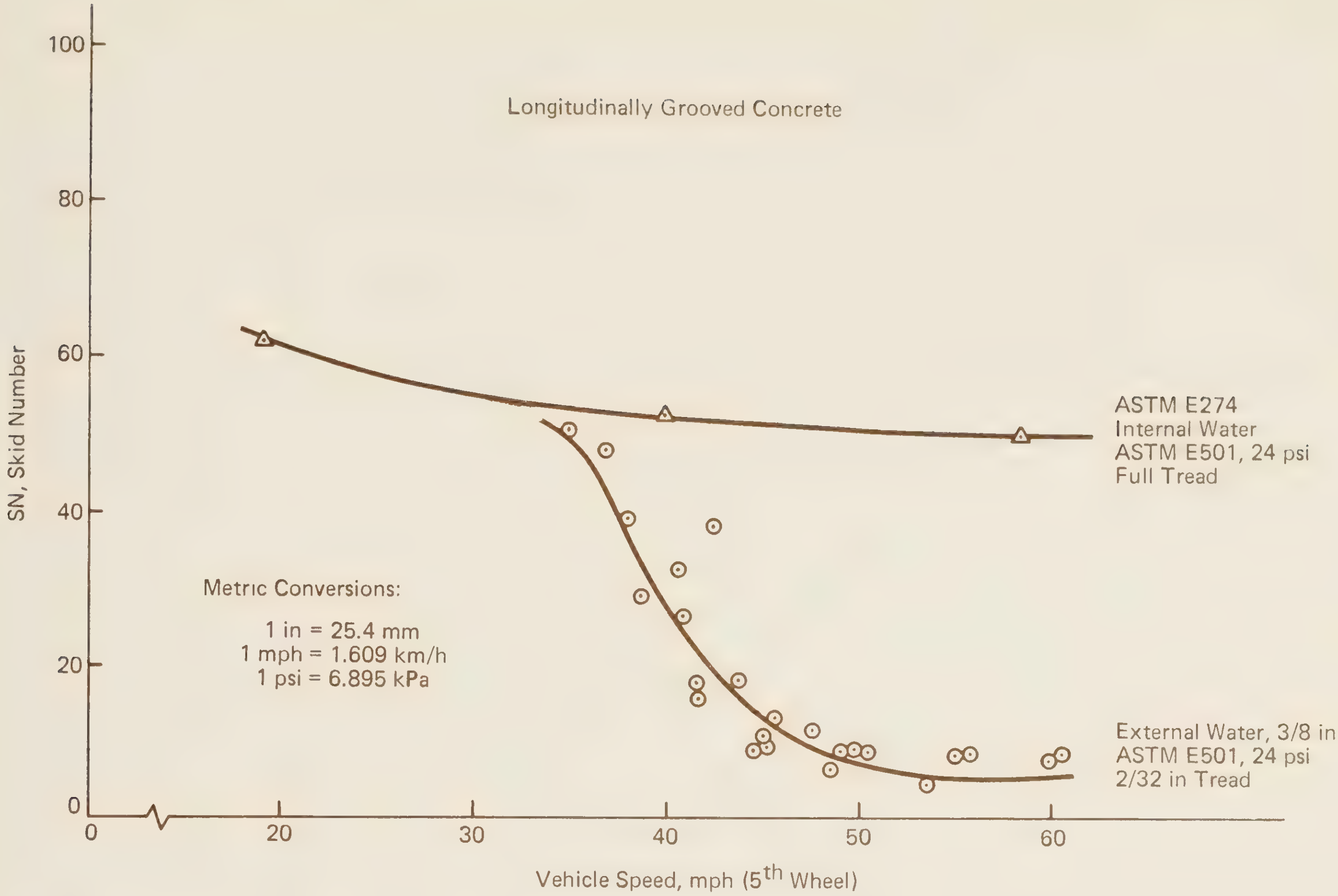


Figure 1.—Locked-wheel skid number versus speed.

Factors That Affect Hydroplaning

Results from conventional ASTM E 274 skid resistance tests on longitudinal grooved, portland cement concrete (PCC) pavement are shown in figure 1. Similar tests with a thicker (9.4 mm [0.37 in]) water film and a smaller (1.5 mm [0.06 in]) tire tread pattern depth of the same type yielded much lower skid numbers, especially at higher speeds. These results, also shown in figure 1, depict hydroplaning. The latter values are not 0, even for full dynamic hydroplaning, because the hydrodynamic drag acts on the tire during testing.

Water escapes between a moving vehicle tire and the pavement as the water is squeezed by the tire load through the coarse pavement texture, through the tire tread pattern, or both. Therefore, a pavement with a coarse texture or finish and tires with adequate tread pattern depth are desirable for wet-weather travel.

Figure 2 illustrates the differences in test results on a coarse texture surface and a medium texture surface for both the radial tire tests and the standard skid tire tests. The radial tire tests were conducted in a water depth of 6.4 mm (0.25 in) above the asperity tops (external source) as contrasted with the E 274 tests with only a 0.5 mm (0.02 in) water film from the internal watering system. There was also a difference in tread thickness. It is clearly evident that skid numbers for the thick water film are much lower than those for the thin water film.

In figures 1 and 2, the skid numbers at test speeds greater than 80 km/h (50 mph) are low for the thick water layers. In fact, hydroplaning may have occurred at speeds less than 80 km/h (50 mph)—nearer 64 km/h (40 mph)—in some of the tests.

The test results in figure 3 were evaluated from wheel spin down percentages. (*1*, *2*) The curves were drawn from an empirical equation obtained by analyzing hydroplaning test results. Water depth, tread depth, tire pressure, and texture depth are constant for all the curves except for the variable that is being studied to determine its significance.

Dynamic hydroplaning increases as the water depth increases, as illustrated in figure 3a. However, a water depth as great as 18 mm (0.7 in) on the highway is rare except in sag-vertical curves, ruts, or other places where the water is ponded. Because the variation of the water depth is small in reality, the change in hydroplaning speed from water depth is relatively small. Nevertheless, a positive depth of 2.5 mm (0.1 in) is sufficient to cause dynamic hydroplaning. If the pavement and tire are smooth, hydroplaning may occur at smaller water depths.

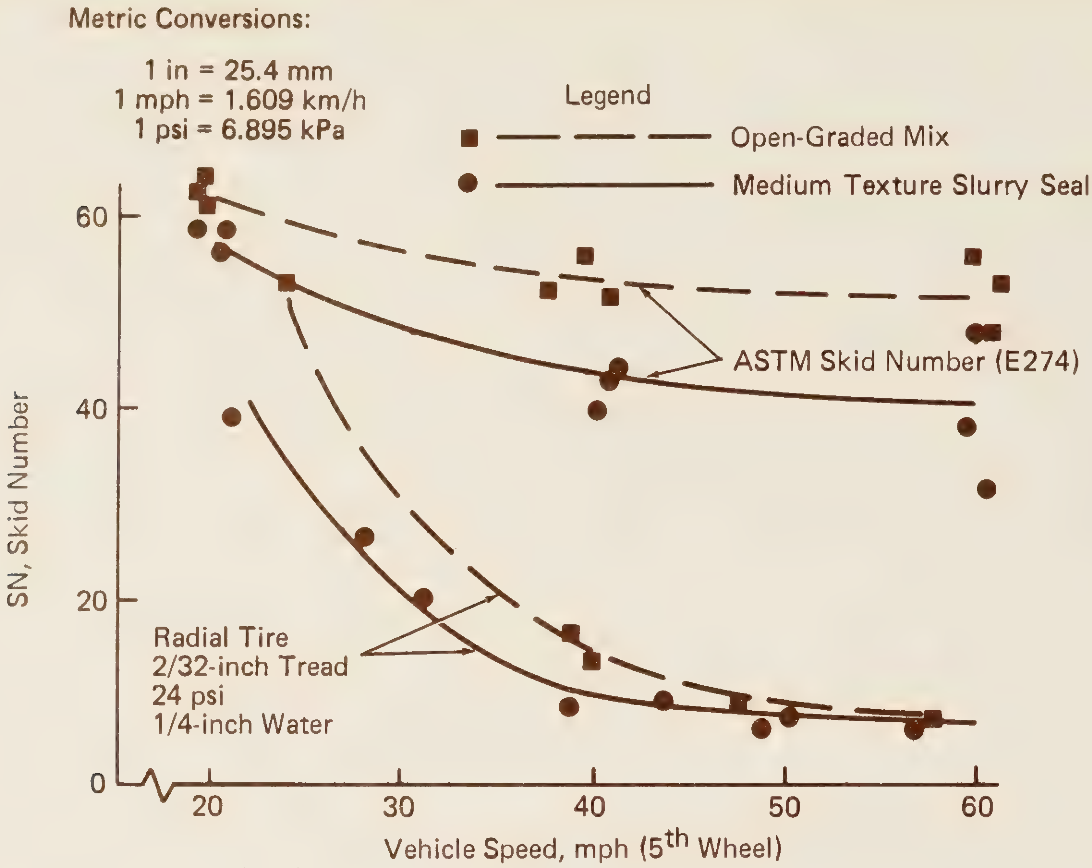


Figure 2.—Test results on coarse- and medium-texture surfaces.

Hydroplaning increases as the tire tread pattern depth decreases (fig. 3b) because the water has less opportunity to escape through the tread pattern. Pattern depth usually has a greater influence on traction and initial hydroplaning speed than either tire inflation pressure or positive water depth. Many States require a minimum pattern depth of 1.5 mm (0.06 in) for wet-pavement travel.

The tendency for hydroplaning can be reduced, however, by increasing the tire inflation pressure (fig. 3c). When the inflation pressure is doubled from 124 to 248 kPa (18 to 36 psi), the initial speed of dynamic hydroplaning is raised more than 16 km/h (10 mph).

Pavement texture is another major factor in hydroplaning. If the pavement texture depth is increased from 0.76 to 3.8 mm (0.03 to 0.15 in), the initial dynamic hydroplaning speed is raised more than 16 km/h (10 mph) (fig. 3d). The deeper pavement texture provides larger escape channels for the water between the tire and the pavement, thus reducing hydroplaning.

The wear rate of a new coarse texture PCC pavement has been stated as follows: A texture depth of 1.5 mm (0.06 in) or greater can be constructed easily and economically with 3.2 mm (0.125 in) metal tines spaced closer than 12.7 mm (0.5 in) apart. Under normal traffic conditions, concrete textures can be expected to wear down approximately 25 to 35 percent during the first 6 months and then remain relatively unchanged for a prolonged period. (*6*) A good quality mortar is assumed.

An open-graded asphalt friction course will provide a coarse pavement texture for bituminous pavements.

Slip Resistance

Figure 4 shows the cornering slip number (CSN), defined as 100 times the ratio of the slip resistance to the tire load, also is larger for a deeper pavement texture. The CSN's are five to seven numbers larger for a texture depth of 1.5 mm (0.06 in) than for a depth of 0.76 mm (0.03 in). The CSN increases with slip angle—the angle between the plane of the wheel and the direction of motion. The CSN also is larger for the transverse pavement texture than for the longitudinal texture.

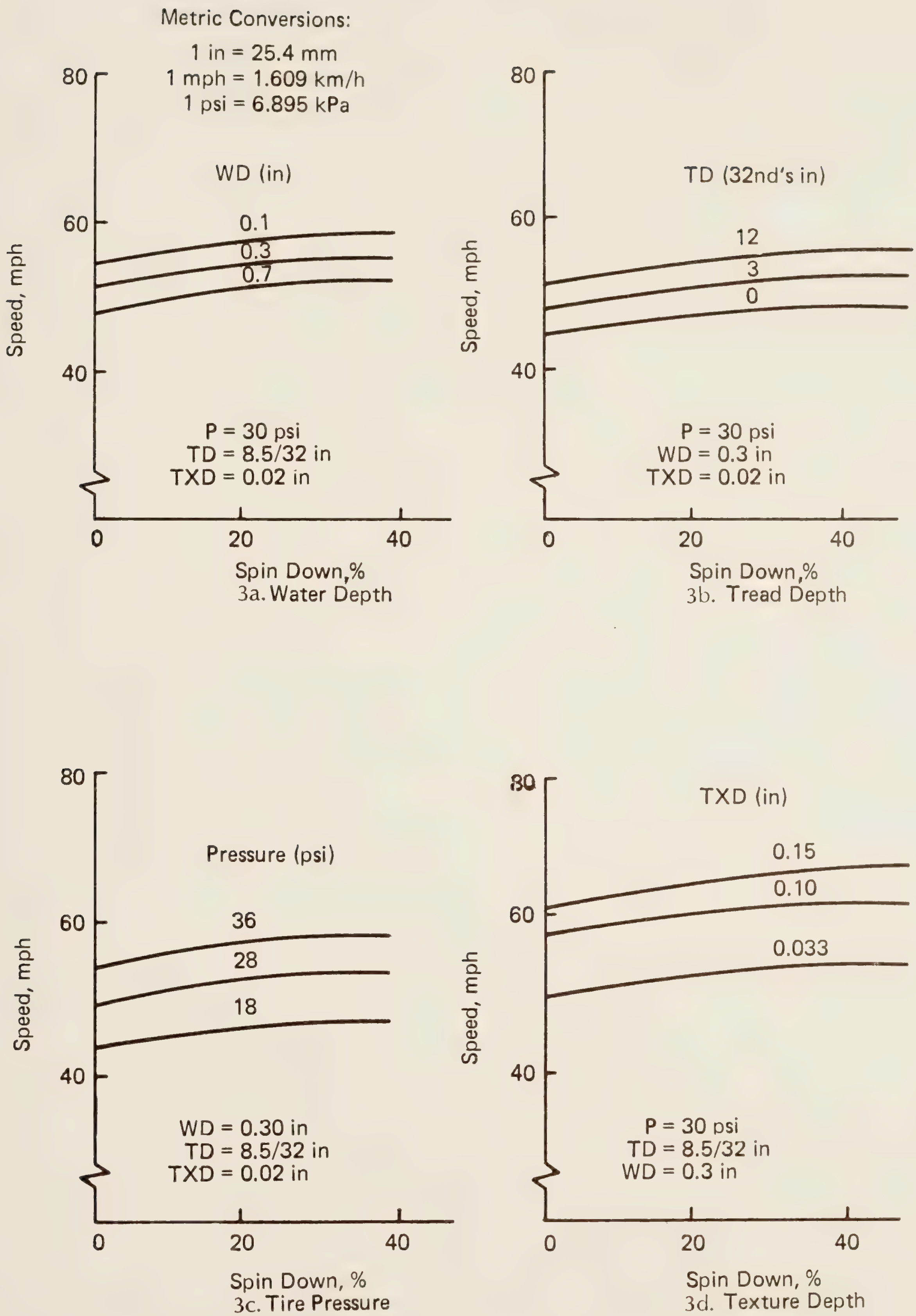


Figure 3.—Influence of four parameters on hydroplaning speed.

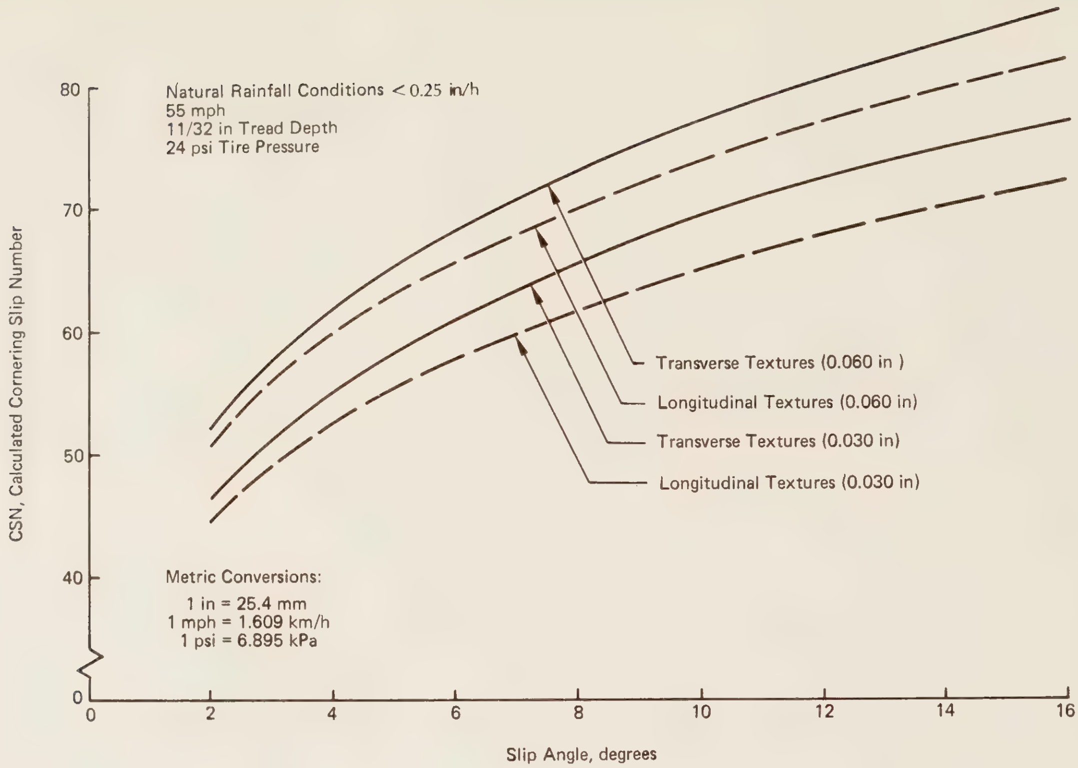


Figure 4.—Cornering slip number versus slip angle.

Transverse texture, alined with the direction of the cross slope, provides better test results than longitudinal textures because as a whole the pavement surface drains better, the shorter relief passageways allow more effective water expulsion between the tire and the pavement, there is less forward motion of water that produces a water wedge to cause hydroplaning, and there is an increased angle of attack over longitudinal grooves.

Effects of Pavement Geometry

Lane changing maneuver

Figure 5 shows the results of a simulation analysis of a 97 km/h (60 mph) vehicle lane change on a crowned two-lane roadway. There is a large, abrupt increase in alining torque at the initiation of the lane change maneuver. The alining torque was computed from simulation analysis. For a 2 percent cross slope, the maximum torque is approximately 158 Nm (1400 lbf·in). As the lane change progresses, the torque decreases and becomes negative even before the vehicle crosses the crown. The friction requirement also increases abruptly at the initiation of the lane change. The initial maximum value is about 0.37. Steeper cross slopes gave larger values, both for the alining torque and for friction.

The friction requirements for entering and continuing on a curve from a tangent are similar to those for initiating a lane change; however, the requirements also depend upon the superelevation and the radius of curvature.

Completing a passing maneuver likewise involves a lane change; however, if another vehicle is approaching, the requirements may be even greater for a hurried completion of the passing maneuver. If there is a thick water film on the pavement surface, the lane changes are even more hazardous.

The requirements in figure 5 are not unreasonable for a dry pavement, and even wet pavements with good skid resistance may meet or surpass them. However, the requirements are greater for a larger cross slope. Furthermore, if the roadway is slick, or covered with snow or ice, there probably will not be enough friction to meet the maneuvering requirements for a 4 percent or greater cross slope at 97 km/h (60 mph).

The effort to steer a vehicle on a 2 percent cross slope is small compared with that necessary to maintain a linear course for cross slopes 4 percent and larger.

Drainage path length

The depth of water on the pavement during precipitation depends upon the rainfall intensity, pavement cross slope, texture depth, and the drainage path length. These relationships are portrayed graphically in figure 6. The curves were obtained from an empirical equation developed by analysis of experimental data. (*1, 2*)

The effect of drainage path length (including multiple travel lanes) can be reduced for a particular rainfall intensity by increasing the pavement cross slope, the texture magnitude, or both.

For example, with a rainfall intensity of 12.7 mm/h (0.5 in/h), a drainage path length of 6.1 m (20 ft) can be accommodated on a 1 percent cross slope with a texture depth of 1 mm (0.04 in). Doubling the drainage path length is acceptable if the cross slope is increased greater than 2 percent, as deduced from figure 6. Alternatively, if the texture depth is increased to 1.5 mm (0.06 in), a drainage path length of 14 m (46 ft) can be tolerated on a 1 percent cross slope. The path lengths are less critical for smaller rainfall intensities.

Wheel path depressions

Greater wear, compaction, and rutting occur in the wheel tracks on a roadway than on other areas. Wheel path depressions interrupt the normal flow of water; excessive depressions can alter the drainage pattern on pavements. Pondir.g of water is prevalent in the depressions and can cause hydroplaning or loss of vehicle control. The ponding is partially or fully relieved through drainage, if the pavement cross slope or longitudinal grade is large enough.

If it is assumed that the minimum cross slope to provide drainage is 0.5 percent and the wheel path depression is 600 mm (24 in) wide, then the following can be deduced from figure 7 (*1, 2*):

$$WPD=S\frac{W}{2}-0.005\frac{W}{2}=(S-0.005)\frac{W}{2}$$

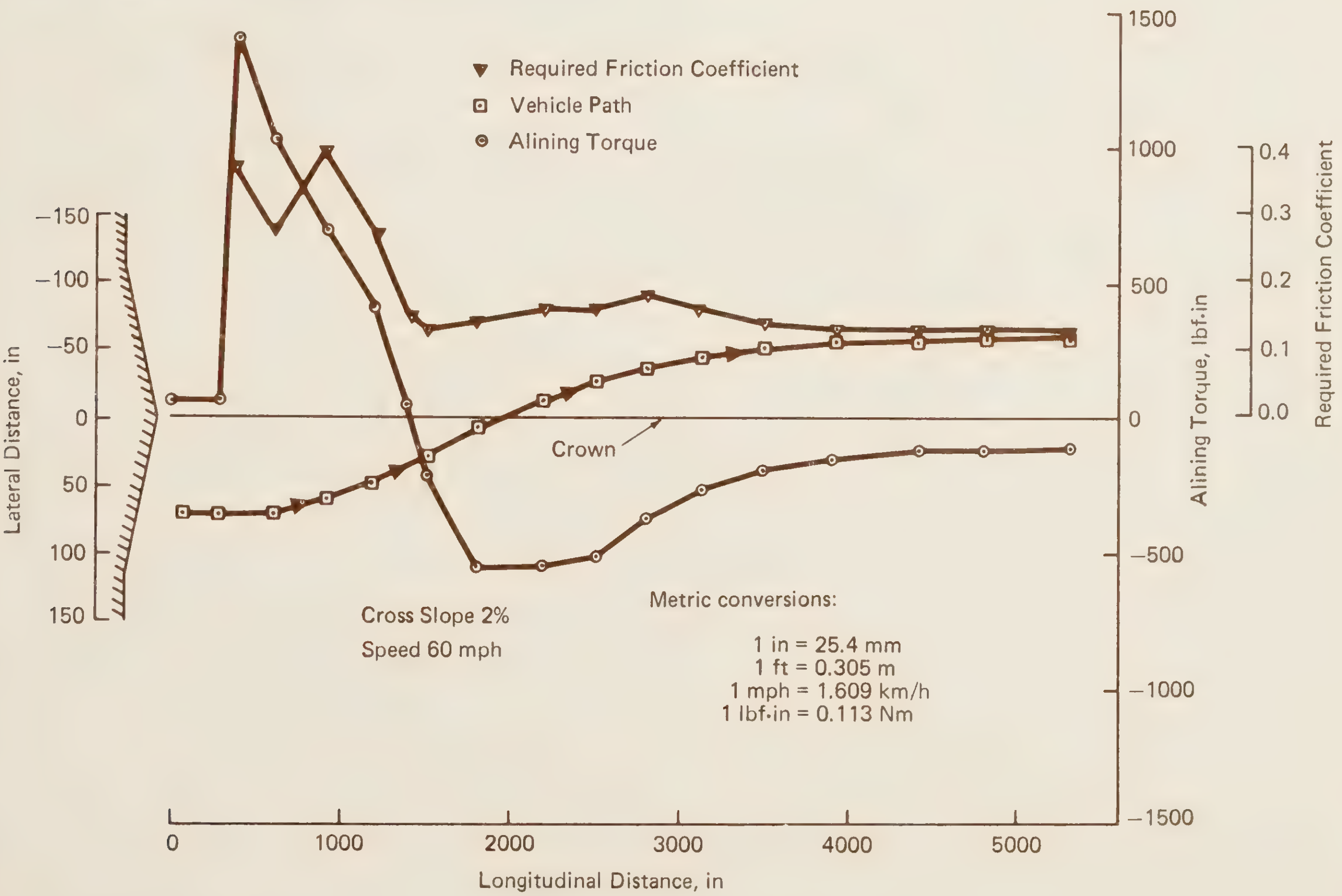
Where,
WPD =Wheel path depression, in millimetres (inches)
S =Cross slope, in percent
W=Width of the wheel path, in millimetres (inches).

The allowable wheel path depressions in table 1, calculated from the above equation, are the admissible values when the influence of cross slope on drainage is considered. Pavement maintenance, resurfacing, or rehabilitation are required when the depressions exceed these depths.

Sag-Vertical Curve Drainage

Vehicle control may be lost at high speeds if the pavement asperities are inundated with water. Removing the water to reduce the water film thickness is the key to decreasing hydroplaning accidents.

Figure 5.—Lane change, torque, and friction requirements. (1)



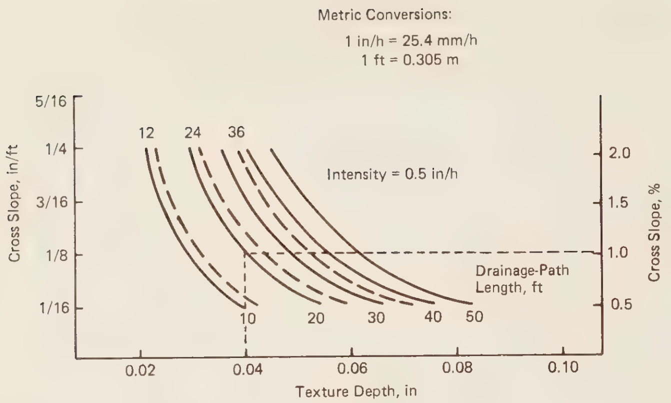


Figure 6.—Acceptable drainage path lengths.

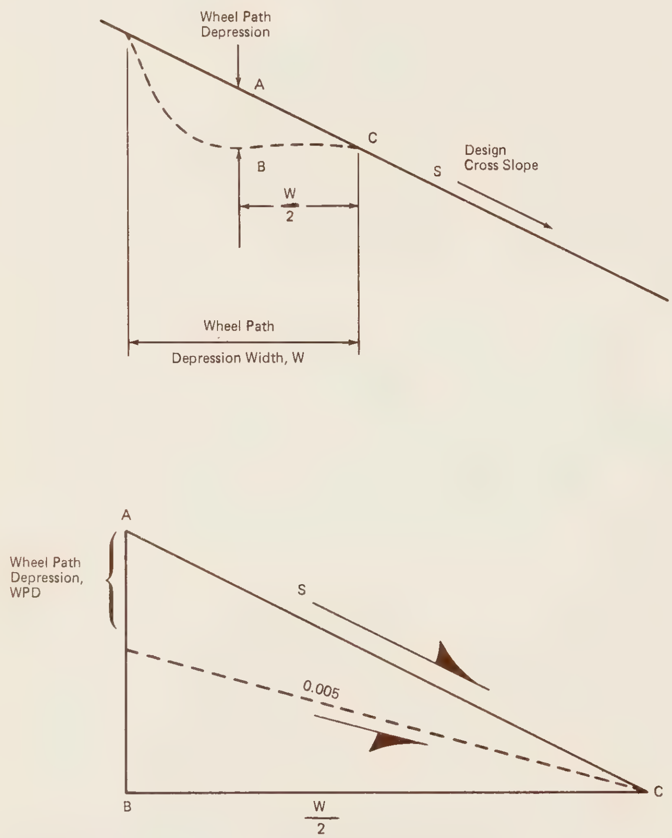


Figure 7.—Wheel path pavement depression geometry.

Table 1.—Allowable wheel path depressions

Pavement cross slope	Maximum wheel path depression
Percent	*Millimetres*
1	1.5
2	4.5
3	7.5
4	10.5

A sag-vertical curve is particularly susceptible to water accumulation. (*1*, *2*) The two grades direct the water runoff toward the bottom of the curve; therefore, this area is especially vulnerable to hydroplaning and reduced traction. The lack of longitudinal grade at the curve bottom requires that the water drain across the pavement.

A storm sewer may be needed at the drainage system's lowest point to remove the accumulating water. The efficiency of the drainage system is critical for reducing the tendency for hydroplaning in this area.

Other Effects of Pavement Texture

Splash, spray, and headlight glare from approaching vehicles can be reduced in wet weather by increasing pavement surface textures. Voids in coarse textures relieve the water pressure between a moving tire and the pavement. The water, therefore, is not ejected as forcefully from these voids into the air as it is from a smooth, wet-pavement surface. (*2*) Open-graded asphalt friction courses are good examples of textures that improve driver visibility by reducing splash and spray.

Textured surfaces diffuse the light from an approaching vehicle to decrease headlight glare. Glare is especially objectionable at night during wet weather. A minimum texture depth between 1 and 1.5 mm (0.04 and 0.06 in) has been recommended to reduce glare. (*7*) Flat spots or polished areas on the surface are undesirable.

Conclusions and Recommendations

- A pavement cross slope of 2.5 percent will facilitate surface drainage, reduce tire hydroplaning, and improve traction during wet-weather travel without hindering vehicle steering.

- A pavement texture depth of 1.5 mm (0.06 in) or greater will improve wet-pavement friction and the cornering slip resistance, decrease hydroplaning tendencies, reduce splash and spray, and diffuse vehicle headlight glare, especially on high-speed highways. Less texture depth is acceptable for low-speed roadways and urban streets.

• Transverse pavement finishes or grooves permit shorter braking distances than do longitudinal grooves. Traffic may reduce coarse texture depths 25 percent or more during the first 6 months. The wear rate will vary with the pavement type and texture characteristics.

• Pavement maintenance or resurfacing is needed when rut depths exceed 6 mm (0.24 in) on pavement cross slopes of 2.5 percent if water ponding is to be avoided. Less rut depth can be tolerated for smaller cross slopes.

• The pavement surface water film thickness, which increases hydroplaning and decreases skid resistance, can be minimized by roadway design, construction, and rehabilitation. The water thickness depends upon the pavement cross slope, texture depth, rainfall intensity, and the pavement surface drainage path length. The drainage path length, which also should be minimized, is a function of the number of lanes and other roadway geometry.

• Drainage facilities should be provided to collect and rapidly remove water from sag-vertical curves to reduce hydroplaning susceptibility and improve traction.

• Tire tread pattern depths should be greater than 1.5 mm (0.06 in) for wet-pavement travel.

• Vehicle tires should be inflated to the maximum recommended pressure to minimize hydroplaning on wet pavements.

• High intensity rainfalls are rare and of short duration; however, hydroplaning or partial hydroplaning may occur from ponded water.

• Traveling speed should be reduced below 80 km/h (50 mph) on wet pavement to decrease the probability of dynamic hydroplaning and improve skid resistance.

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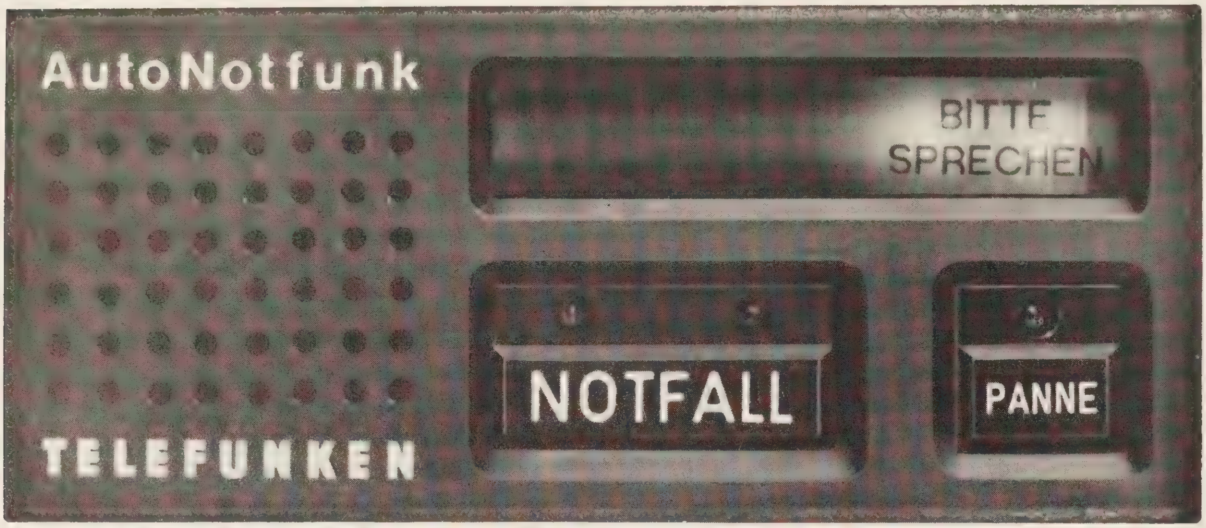
Glenn G. Balmer is a highway research engineer in the Pavement Division in FHWA's Office of Engineering and Highway Operations Research and Development. He is involved in research on the surface properties of pavements with emphasis on skid accident reduction, minimization of hydroplaning, and diminution of road roughness. Before joining FHWA, Mr. Balmer was a senior development engineer with the Portland Cement Association in Illinois.

Bob M. Gallaway is a professor and research engineer at Texas A&M University. He formerly was head of the Highway Materials Division of the Texas Transportation Institute and the Materials Science and Materials Engineering Division of the Civil Engineering Department. Mr. Gallaway is involved in research on tire-pavement interactions, skid resistance of pavements, maintenance methods for improved pavement surface properties, and sulfur-asphalt binder systems.

[4]Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

Autonotfunk—Emergency Voice Radio in Automobiles:



The System and Some of the Problems

by
Bernhard Schlag and Gernot Riediger

An in-car emergency call system can reduce the time between the occurrence of an emergency and its reporting and thus improve the survival chances of injured people. This article describes the emergency voice radio system Autonotfunk developed in the Federal Republic of Germany. Some of the nontechnical problems associated with the system and test and evaluation of the system are discussed.

Introduction

The time lag between the occurrence of an emergency and its reporting is particularly difficult to reduce outside urban areas where the lack of reporting means and the long distances for ambulances generally delay aid times considerably. A first approach to shortening the reporting time was installing stationary emergency telephones on heavily traveled rural roads. Nearly all autobahns are equipped with emergency call boxes at average intervals of 2 km (1.2 miles). In addition, about 1,100 emergency call boxes on federal and state highways cover about 4 000 km (2,500 miles) (about 3.9 percent) of this highway network. However, an optimally equipped stationary reporting system of this kind does not reduce reporting time in emergencies where victims are not capable of looking for the next call box to report the emergency. Because of this and other drawbacks involved in stationary reporting facilities, the possibility was explored of reporting emergencies by in-car communication techniques (for example, car telephone or citizen band radio). However, the advantage of direct access, for example with the citizen band radio, is confronted with the disadvantage of not being certain the call will be received at the right place.

Autonotfunk

The emergency voice radio system Autonotfunk (ANF), developed by AEG-Telefunken on behalf of the Federal Ministry for Research and Technology, combines the advantages of the mobile reporting system (direct access independent of time and place) and the stationary reporting system (being sure the report is received immediately by the appropriate responding agency). In addition, the ANF also provides oral communications between the communications center and the caller and automatic location identification by compass bearing.

The main objectives of the ANF are to shorten the time between the occurrence of an emergency and the arrival at the scene of the help required, reducing not only the reporting time and the time needed to locate the incident but the consequences of the emergency as well; improve the quality of messages by using available communication facilities, thus reducing delay by dispatching appropriate assistance; and improve communications for reporting disabled vehicles, thus reducing the possibility of additional damage to the stranded vehicle or other vehicles.

The ANF consists of the following four components:

Mobile transceiver in the automobile, either integrated into the car radio (fig. 1) or installed as a separate part. Emergency messages are transmitted by pressing the SOS button or the BREAKDOWN button. In addition, automatic release can be provided.

Fixed relay stations that provide reception, bearing, and transmission facilities. The stations are installed to receive distress calls from the automobile, locate the source of the call by compass bearing, and transmit the call to the nearest emergency medical service (EMS) communications center. About 4,000 relay stations spaced about 8 km (5 miles) apart would be required for effective coverage of the total area of the Federal Republic of Germany.

An *EMS communications center* (fig. 2) is the central location for processing and acknowledging emergency calls and providing two-way communication with the caller. To provide adequate coverage, approximately 250 existing EMS communications centers in the Federal Republic of Germany would have to be equipped with the facilities to handle ANF calls.

A *disabled vehicle station* as communications center to handle breakdown calls and to which breakdown calls would be automatically transmitted. These centers would be operated separately from the EMS communications centers. Additional information about the location and circumstances of the emergency could also be requested by radio voice with the caller. These systems would have to be established all over the Federal Republic of Germany or be integrated into available facilities.

Figure 3 shows the integration of the ANF into the existing emergency call system and figure 4 displays the ANF network.

Figure 1.—ANF car set integrated into the car radio.


Figure 2.—ANF operating and control desk at EMS communications center.

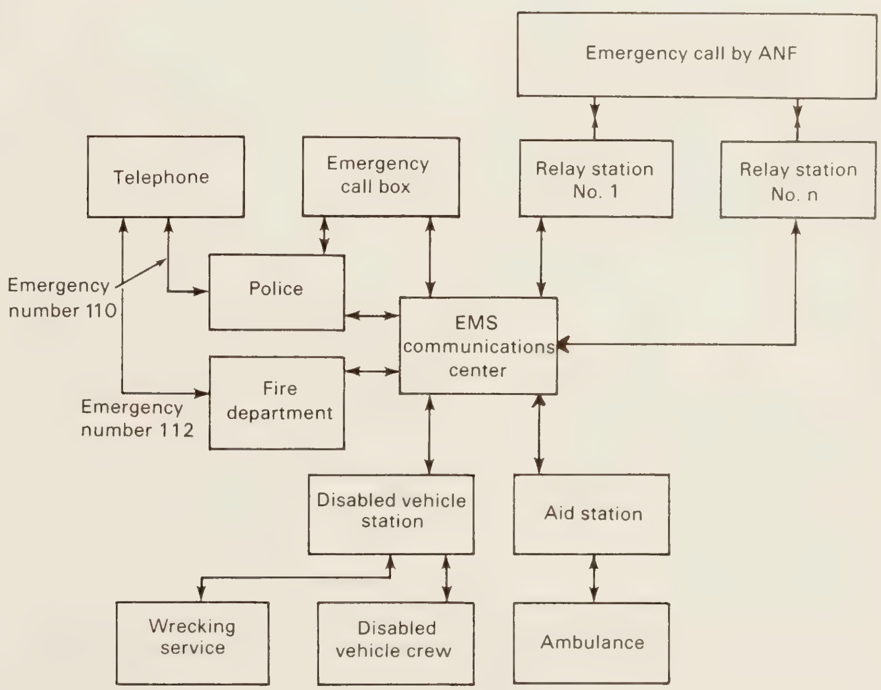


Figure 3.—ANF integrated into existing emergency call system.

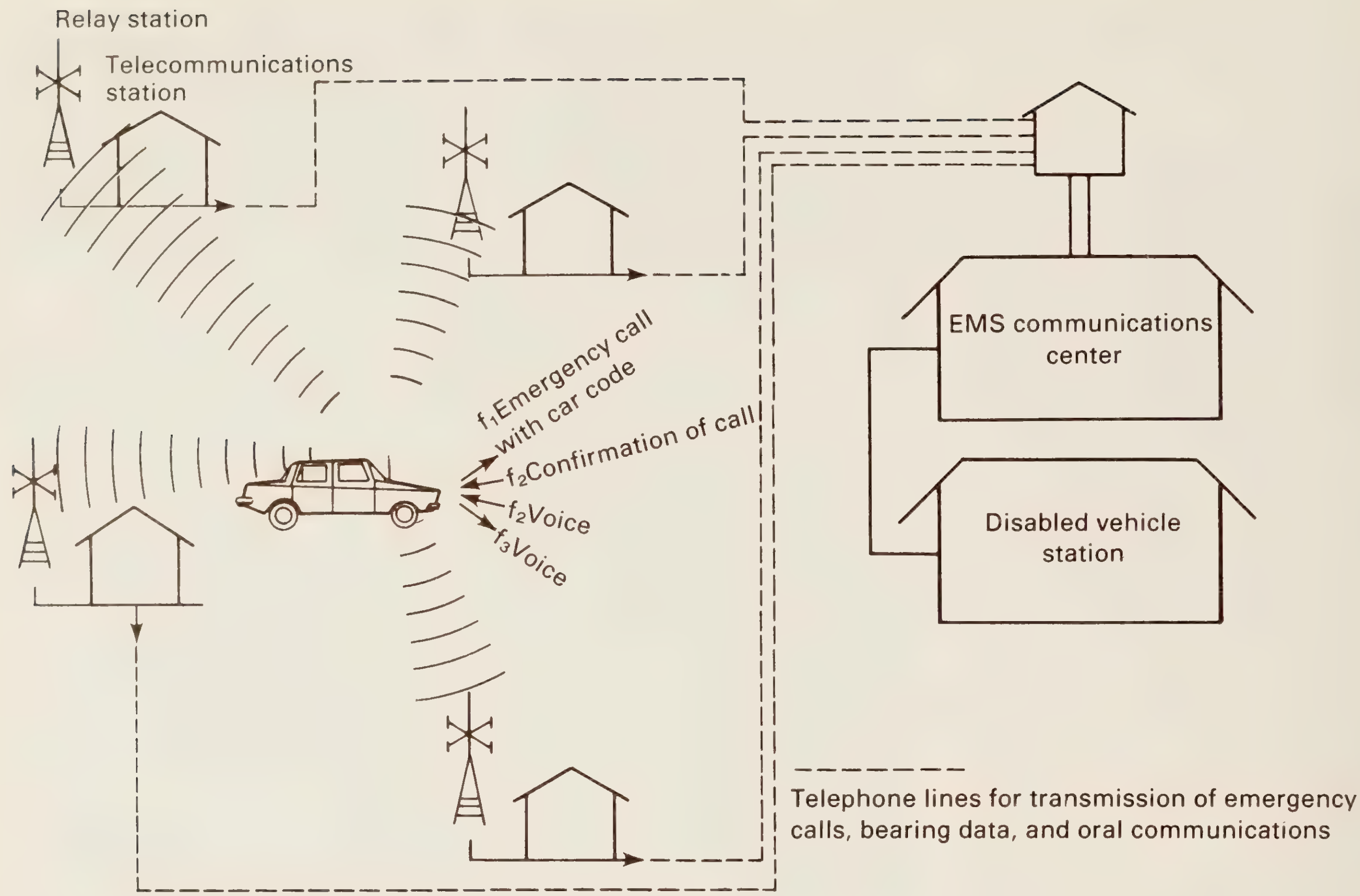


Figure 4.—ANF network.

Effects of the ANF

The ANF appears to be a valid system; however, its effects would be felt by the entire rescue and ambulance service. Therefore, the direct and indirect effects the system will have on potential users and those responsible for operating the service need to be studied, as do the associated costs.

With the introduction of the ANF, reporting methods would perhaps be so drastically improved that a radically different and increased EMS workload would have to be handled. Emergency incidents probably would not only be reported by the party concerned but also by drivers not involved in the incident.

The ANF would have to be integrated with a set of already rather complex activities at the EMS centers. At these centers the dispatch and control of aid teams and vehicles are coordinated using special technical aid systems. The ANF system will require additional telephone lines (to connect with the relay stations and disabled vehicle centers) and additional input and output units for the computer at the EMS center. Moreover, the ANF system will require communication lines whose admissibility in the Federal Republic of Germany is yet subject to discussion regarding the legal aspects of telecommunication.

It is difficult to determine whether the ANF will ease the workload, make better use of available capacities, or result in overload conditions at EMS centers. The presumably more frequent calls to the staff of the EMS center would have to be compared with the advantage of being able to communicate with callers and react more effectively to what is required in each case. Among other things, the extent to which the operations of an EMS center will change because of the introduction of the ANF, the resulting demands on the personnel, and the effects on dispatching and initial care will have to be studied.

Abuse of the system or use for purposes other than the ones for which it was conceived also will have to be studied. It can be foreseen that the ANF in the automobile, as is the case with the emergency call boxes, will not be used solely for reporting medical emergencies. Probably, disabled vehicle calls, considered secondary in importance at ANF receiving stations, will outnumber "true" emergency calls. Permissible uses will have to be defined. Calls for information will have to be controlled. Assistance might even be requested in nonemergency cases just because of the availability of the system. Inadvertent or unintentional use or even intentional abuse must be prevented by technical and organizational means. To put a call through, the SOS button has to be pressed twice or once for at least 3 seconds. In addition, the caller or automobile owner can be identified by way of a code system automatically added to the message.

Preconditions and Problems for Users and Agencies Operating the ANF

In general, users of the ANF primarily will be automobile drivers and passengers, but the system also may be used by all other road users or those happening to be at the location. Users' attitudes toward purchasing the ANF car equipment and correct usage regarding procedural regulations, including operating rules and billing procedure, are vital. User acceptance of the system, which will essentially affect system expansion and effectiveness, depends on system costs, awareness of the need for shortening rescue times, and potential uses.

Acceptance of the ANF and the resulting organizational and procedural forms could be one of the major problems for the agencies operating and responsible for the ANF system. Standardization of the technical equipment at EMS centers and more uniform access, operation, and billing procedures are needed. However, the ANF system must adhere to the basic principles underlying the present organizational structure of the EMS.

Organizational prerequisites for the establishment and operation of the ANF at disabled vehicle stations must be provided in collaboration with the various organizations now offering disabled vehicle services.

Further, legal problems such as conditions for licensing the technical equipment or questions of radio frequencies and broadcasting rights must be solved.

On the whole, the ANF will force the different personnel responsible for the service to develop jointly acceptable organizational policy rules, thus deciding what the future concept of the EMS and disabled vehicle service will be. This is even more evident when it comes to considering the problem of financing. Different approaches are conceivable, ranging from offering the system at no cost for the user to charging for limited periods of use or individual emergency calls.

Also, data processing of identification codes and accounting procedures may raise data protection problems. Moreover, there are questions concerning insurance and liability, so the prerequisites for and consequences of equipping cars with ANF systems need to be thoroughly examined.

The problems mentioned indicate a wealth of prerequisites to be settled, all of them potentially influencing the technical design and the costs of the system.

Distributing and apportioning the costs for the ANF system among the public authorities, the health insurance companies, and the users involved will be as difficult as distributing costs according to service items. Information is not yet available on general operating costs, such as personnel expenses, costs of organization and accounting, or the costs of financing. A clearer idea of cost items will evolve when a precise concept of organization is developed.

Because high system costs are expected, other applications of the ANF virtually are required. Possibilities include linking the system with automatic route guidance systems, regional radio warning, or the rescue services of industrial companies or disaster control services.

Test and Evaluation

Because of the great expectations for the ANF to improve reporting and aid conditions in emergencies, and because of the many problems associated with the integration of the technical system into the EMS, the Federal Ministry for Research and Technology (BMFT) decided that indepth studies on the ANF should be undertaken. The German Research and Test Establishment for Air and Space Travel (DFVLR) was responsible for the project, with the Federal Highway Research Institute (BASt) responsible for coordinating and managing the nontechnical research. The BASt, in turn, established a project group to work out a comprehensive concept of how to approach the problem scientifically. (*1*)[1]

The technical data necessary for assessing the equipment and system design of the ANF will be acquired from a field test in the area of Darmstadt. This field test will provide information on which to base the final technical specifications. AEG-Telefunken will carry out the technical performance tests, equip the test area (relay stations and communications center), and provide the test sets (ANF car sets) and maintenance and repair services.

The nontechnical research to gather information on the problems for users and operating agencies includes 14 individual projects following each other in three stages (fig. 5), in connection with other related research projects of the BMFT, the DFVLR, and the BASt.

[1]Italic number in parentheses identifies reference on page 24.

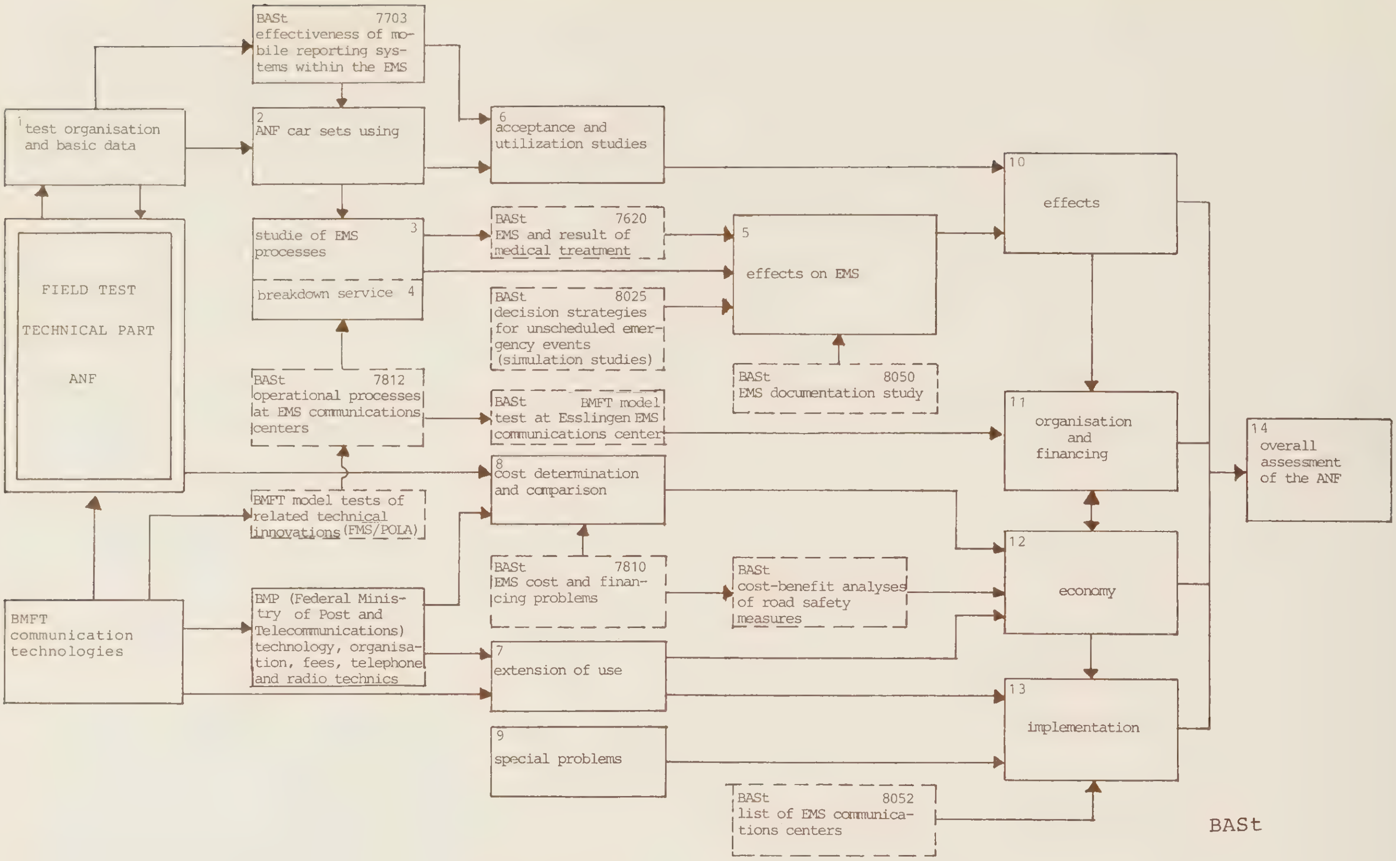


Figure 5.—Project linkage.

Timetable for Possible Implementation of the ANF

Based on an optimistic estimate of when the ANF could be operative, the following timetable has been prepared.

Research will continue until August 1983 and will be followed by the preparation of the technical specifications for the ANF, expected to take about 18 months. If a decision for implementation of the ANF is made at the time the specifications are available (in view of the changing social and economic framework, this is likely to be one of the most critical phases of the timetable), the technical development of the system up to the production stage could be started at once. The production of ANF car sets, relay stations, and communications center equipment parts for the ANF system is expected to follow in 1986, at the earliest. If, among other things, sufficient production capacities and funds are available for the system, ANF could become operative—presumably on a regional basis at first—at the beginning of 1987. Once operation has started, it is reasonable to expect that a more widespread use of such an innovation will take at least 10 years.

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Bernhard Schlag, a psychologist specializing in accident research, is responsible for publishing research reports and coordinating and managing the nontechnical research on the ANF at the Federal Highway Research Institute. Dr. Schlag also lectures on psychology at the University of Essen, Federal Republic of Germany.

Gernot Riediger is an economist specializing in road safety research. He carried out economic analyses at the Federal Highway Research Institute and was responsible for planning research tasks in the field of emergency medical service. Since 1982 he has been working at the Federal Ministry of Transport and is involved in the areas of road safety and emergency medical service.

Editor's Note

Various individual invehicular communication concepts to provide motorists with guidance, advisory, and emergency information have been investigated over the past years in the United States as well as abroad. Following are brief descriptions of some of the systems considered or developed in the United States.

Highway Advisory Radio (HAR):[1] Provides highway and driving related messages to the motorist using the existing AM radio receiver. HAR stations transmit on either 530 or 1610 KHz. Use of these frequencies is authorized by the Federal Communications Commission. HAR has been installed in more than 30 States.

Automatic Highway Advisory Radio (AHAR): Will provide the motorist with a radio receiver that automatically tunes the motorist's radio to driving and travel related information and returns the radio to its initial broadcast after the travel message has been repeated twice. The receiver also has a priority message hierarchy and an automatic override for emergency vehicle transmission. The AHAR system will overcome several limitations of the manual HAR system.

Experimental Route Guidance System (ERGS): Designed in the late 1960's to provide the motorist with directional guidance. In its initial static configuration, ERGS consisted of an invehicular display, a destination entry device, and a roadside communication and control unit. When starting a trip, the motorist would enter a five-letter destination code word via thumbnail switches into a console mounted in the vehicle. As the vehicle approached an instrumented intersection, its destination would be transmitted via near field radio to the roadside unit, which would then transmit instructions back to the vehicle display. The system was tested and evaluated in the Washington, D.C., metropolitan area.

Citizens Band Radio:[2] Popular for its entertainment value as well as its use in emergencies as a motorist aid and communications system. The CB/Aids system establishes an automatic telephone patch from remote repeaters to an emergency responding agency. The system was tested and evaluated in DeKalb County, Ga.

Michigan Broad Emergency Assistance Radio (BEAR): Uses CB radio for motorist aid and consists of 10 remote repeater stations along Interstate 96 between Detroit and Grand Rapids. The stations link via dedicated leased telephone lines to the Michigan State Police in East Lansing. A motorist with a CB can report problems on channel 9. The message is picked up by a remote station then sent by telephone lines to a volunteer police dispatcher who relays the message to an officer along the highway or to a State police substation.

National Emergency Aid Radio (NEAR): An effort by the National Highway Traffic Safety Administration (NHTSA) to promote the use of CB radio for emergency situations. NHTSA provides funding and training programs to increase the citizen's role in highway safety and make various emergency services available. Monitoring is performed by public safety agencies, organized volunteer groups, motor carriers, professional driver groups, and individual citizens. Coordination is handled by public safety, law enforcement, or other government agencies that wish to participate.

Safety Hazard Advanced Warning System (SHAWS): Could warn motorists of hazardous travel situations through a system based on modern electronics and microprocessor technology. A simple, low powered transmitter is placed near the roadway shoulder upstream from a hazard. A receiver is required in the vehicle to receive, process, and display transmitted warning information. The idea of the system is to extend driver perceptions, allowing the motorist, in effect, to see around obstacles and over hills, to maneuver accurately, judge unsafe speeds, and be alerted to hazards.

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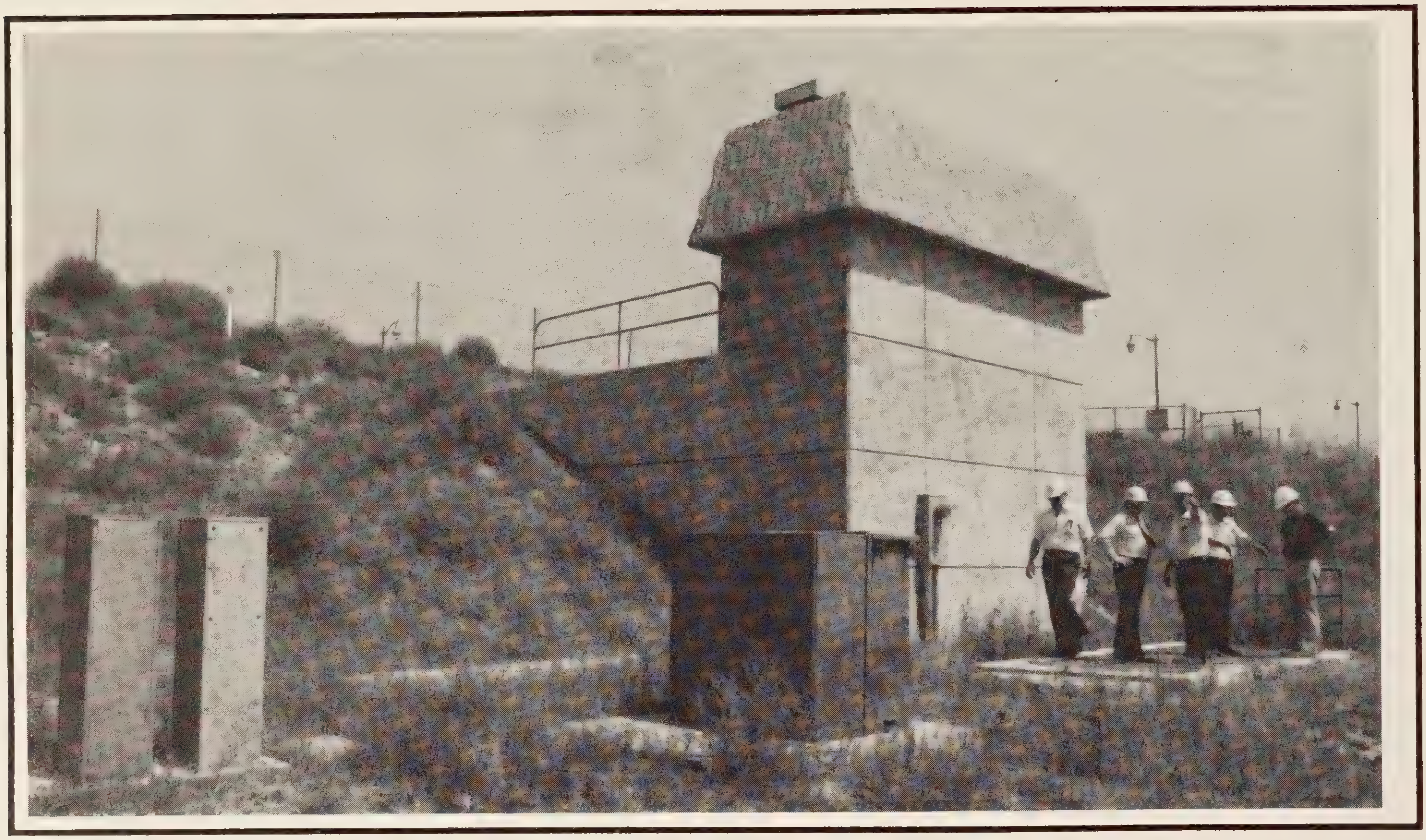
[1]Additional information on HAR can be found in the following reports. Each is published by the Federal Highway Administration, Washington, D.C., and is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161. "Highway Advisory Radio User's Guide," FHWA/RD-80/166, May 1981 (PB 82 123662); "Highway Advisory Radio System Design Guidelines," FHWA/RD-80/167, May 1981 (PB 82 123654); "Systems Analysis and Design Guidelines for Highway Advisory Radio," FHWA/RD-80/176, May 1981 (PB 82 123613); and "Highway Advisory Radio Message Development Guide," FHWA/RD-82/059, October 1982.

[2]Additional information on systems using CB radio can be found in the article "CB Motorist Aid—State of the Art," by Frank J. Mammano, *Public Roads*, vol. 44, No. 3, December 1980, pp. 123–129.



Highway Stormwater Pumping Stations

by

William F. Lever

Introduction

The highway engineer usually is familiar with hydraulics and drainage but may regard the design of pumping stations as a specialized field. "Design Manual for Highway Stormwater Pumping Stations," a two-volume report, presents complete and current procedures for every aspect of stormwater pumping station design.[1] Cost-effective design procedures such as the use of storage to reduce peak flow rates and associated pump capacities are emphasized. Detailed information is provided on kinds of pumps, pump motors, natural gas engines, electrical systems, and wet-well and piping systems. Discussions are included on pump station siting, hydrologic and hydraulic considerations including pump cycling, and operating details important to maintenance personnel. Seven appendixes listing manufacturers of pump and pump station equipment, details of pump and engine testing, suggested maintenance management procedures, and a bibliography comprise Volume II of the manual.

The manual describes different kinds of stormwater pump stations and fully explains their features and equipment. To gather data for the manual, State highway agencies were invited to submit information on their installations. Local government agencies also provided valuable data. Field surveys were made in four large urban areas with different climates and extensive freeway systems to assess design criteria and operation and maintenance practices. These four areas were Detroit, Mich., Houston, Tex., Los Angeles, Calif., and Phoenix, Ariz. Each area has several pump stations constructed in accordance with standards adopted by the governing agency, though designed for the appropriate pumping capacity and head required to suit individual conditions.

[1]Report No. FHWA-IP-82-17, published by the Office of Implementation, Federal Highway Administration, is available for $9.50 (Vol. I) and $7.50 (Vol. II) from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock Nos. 050-001-00254-0 and 050-001-00253-1).

The standards and kinds of stations differed greatly from one State to another, but all were effective and generally interchangeable. A good standard design developed for one area of the Nation could be used in other areas, even those with greatly different climatic conditions. However, a matrix selection process has evolved so that special local features or conditions may be considered when selecting a kind of station.

Kinds of Station

There are two basic kinds of pump station—the wet-pit and dry-pit—and variations of each kind. In the wet-pit, pumping equipment is submerged or suspended into a wet-well containing the stormwater to be pumped out. In the dry-pit, the pumping equipment is in a separate dry-well connected by piping to the wet-well containing the stormwater.

Most pumps are driven by electric motors, but internal combustion engines also are used extensively as drivers. Pump motors on engines start and stop automatically according to water level in the wet-well or dry-well.

Most pumping stations are wet-pit with vertical mixed-flow or propeller pumps suspended from an upper level into the stormwater. The pumps must be adequately submerged and there must be adequate storage so that pumps do not cycle or start and stop at short intervals causing electric motors to overheat, which could cause subsequent damage.

The pumping capacity of different stations may vary considerably, from less than 0.3 m^3 (10 ft^3) per second to 8.5 m^3 (300 ft^3) per second or greater. The discharge head also will vary according to conditions; however, 6.1 to 7.6 m (20 to 25 ft) is consistent with a grade separation or overpass.

The dry-pit station developed and adopted as a standard by California has a large concrete storage box (or wet-well) usually constructed below the pavement adjacent to the station. This ample storage reduces pumping capacity; only two or three small horizontal nonclog centrifugal pumps installed in the dry-well (fig. 1) are required. A circular wet-pit station with electric motor driven vertical pumps (figs. 2 and 3) has been adopted as a standard by Detroit, Mich. A concrete cylinder is excavated so that it sinks into the ground to the desired level in this caisson-type construction. The relationship among frontage road, side slope, pump station, and highway pavement can be seen in figure 4, which shows a large caisson-type wet-pit station.

Collection Systems

The collection system of pavement, ditches, channels, and underground drains upstream of the pump station must always be considered with the station because both are interdependent. Catchment area, runoff, time of concentration, and system storage computations

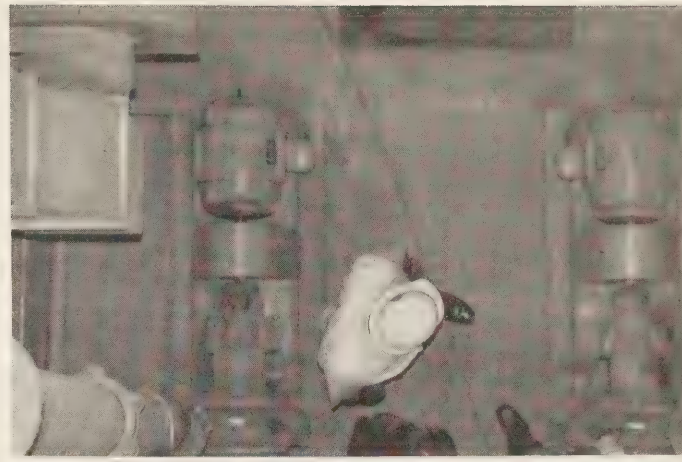


Figure 1.—Centrifugal pump in dry-pit station.

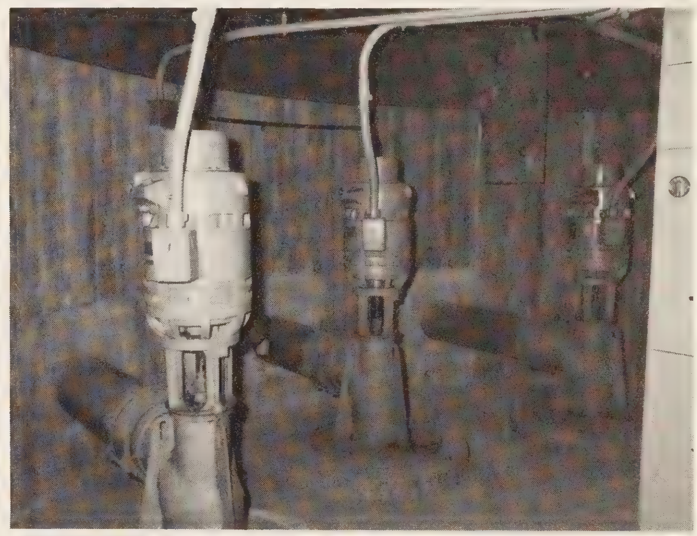


Figure 2.—Motors driving vertical pumps.

Figure 3.—Caisson-type wet-pit station.

Figure 4.—Large wet-pit station.

and the inflow hydrograph should be available to the pump station designer. Storage can contribute to reducing the size and capacity of pumps and drivers and therefore reduce costs.

Pumping Machinery

Five basic kinds of pump are used for stormwater pumping. End-suction centrifugal pumps are shown in figure 1. The vertical pump most often is used in the wet-pit. The upper portion, discharge, and motor of this pump are shown in figure 2. Below the floor slab is a vertical column submerged in the water. The propeller rotates in a bowl near the bellbottom. Driven by a vertical shaft from the motor, the propeller drives water up the column and out through the discharge.

Five submersible pumps are housed about 9 m (30 ft) below the level shown in figure 5. Each submersible pump has an electric motor that is attached directly to the pump casing and that operates below the water surface. Each pump has a flexible discharge hose or other means so that the pump can be raised out of the pit for inspection and maintenance.

Figure 5.—Caisson for submersible pumps.

Angle flow pumps are set in the bottom of a dry-well and have an elbow beneath each pump to take suction from an adjoining wet-well. A vertical motor at or near grade drives each pump through a vertical shaft. This kind of pump commonly is used to pump sewage flows.

The fifth kind of pump, the screw pump, is suited primarily to low lift or heads and is set on a slope compatible with the side slope of a depressed highway section. It may be driven by electric motor or gas engine. This kind of pump has been used for the intake works of wastewater treatment plants.

Heavy, stationary gas engines drive the pumps. Figure 6 shows a standard design developed by Texas in which a combination drive is used. The engine in figure 7 drives a vertical pump through a right-angle gear drive. An electric motor is mounted above the gear head so that either the engine or the motor can drive the pump, according to which is engaged. This increases reliability by eliminating problems caused by electrical power failure. Figure 8 shows the exterior of

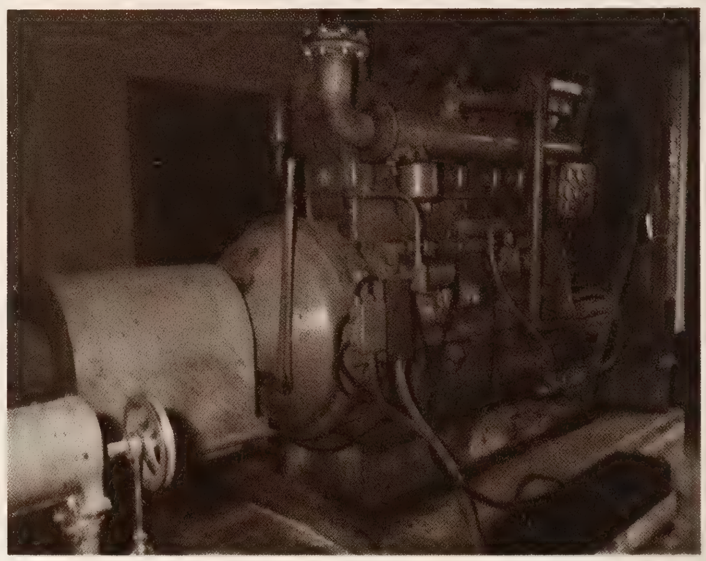


Figure 6.—Natural gas engine.

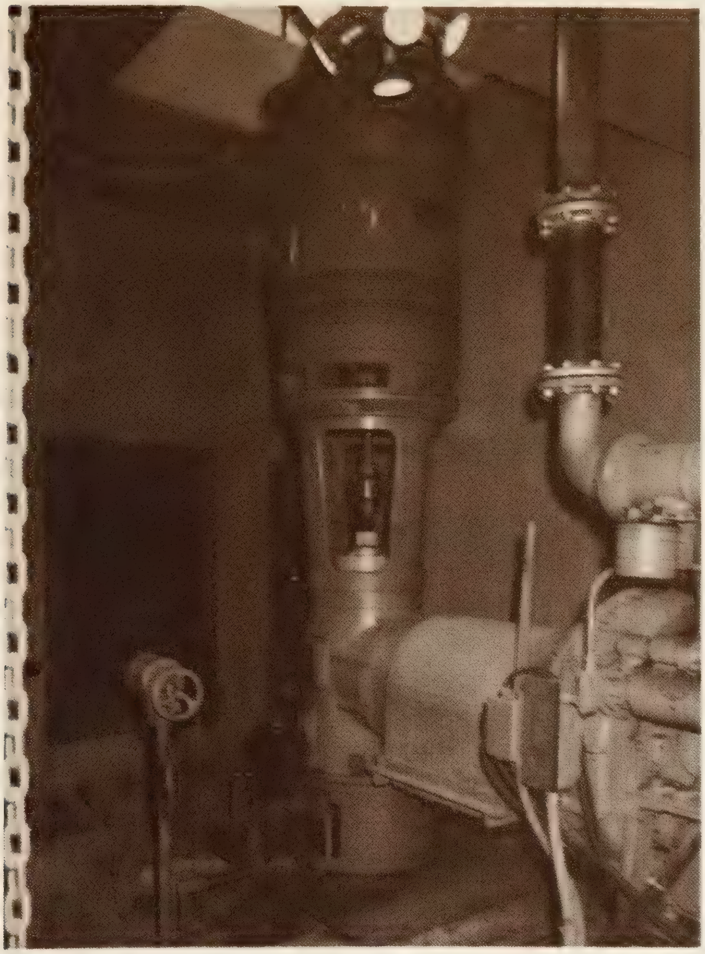


Figure 7.—Combination drive for vertical pump.

Figure 8.—Texas standard wet-pit station.

the station. The air that the fan has moved through the radiator for engine cooling passes through the large louvers. A storage tank for liquified petroleum gas (LPG) also can be seen. LPG stored on site is an alternative to piped natural gas which is not always available in rural areas.

Electrical Systems

The same Texas station has overhead service at high voltage (fig. 9), with pole-mounted transformers that reduce the voltage to 480 volts, the requirement for 250 horsepower motors. A higher voltage may be used where motor horsepower is higher; the vertical pumps in the station shown in figure 4 have 700 horsepower motors and 2,400 volts service. However, motor horsepowers usually are compatible with 480 volts service. An underground service is preferred to the overhead wiring shown in figure 9.

Various control systems for automatically starting and stopping the motors and engines and the basics of electrical requirements and features are described in the "Design Manual for Highway Stormwater Pumping Stations." Improper design may require special and expensive equipment to start the motors.

Figure 9.—Power supply to Texas station.

Construction Details

Because pump station construction involves many details not widely used or recognized, the manual describes at some length details such as architectural features and the external appearance of the station. Poured concrete and concrete block are the usual materials; metal buildings may have some application. Ventilation, water supply, and plumbing requirements are discussed. Hoists and other handling devices for trash and debris removal are described and illustrated.

Retrofitting

Many agencies have one or more existing pump stations that perform inadequately or present a maintenance problem. Recognizing this situation, the manual gives some examples of how existing stations have been modified to upgrade performance. Water flow and hydraulic conditions may be improved by structural changes, particularly in rectangular wet-pits with vertical pumps. In other cases, replacing existing pumps with different kinds of pumps improves the station. Figure 10 shows a retrofitting where an existing wet-pit station serving a railroad underpass was converted to a dry-pit station with horizontal centrifugal pumps.

Figure 10.—Wet-pit station retrofitted to dry-pit.

Summary

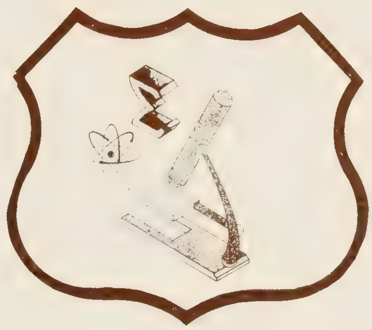
Stormwater pumping stations, located throughout the U.S. highway system, are necessary where gravity drainage flow is impossible or uneconomical. Most pumping stations throughout the Nation are unobtrusive and reliable, primarily because they are well-designed and equipped. However, malfunctions do occur, in some cases because of unsuitable design. The two-volume manual discussed in this article provides guidelines, criteria, and specifications intended to improve understanding of a subject made more complex by the multiple choices available for the design of highway stormwater pumping stations.

William F. Lever, a civil and structural engineer, is involved in the design of stormwater pumping stations. His engineering firm has been involved in projects under the jurisdiction of the Los Angeles County Flood Control District, and the firm was contractor to FHWA for the manual on pumping stations described in this article. Mr. Lever also was involved in the design and construction of the North Harbor storm drain pump station for the Port of Long Beach, Calif.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Federal Highway Administration, Offices of Research, Development, and Technology. The Office of Engineering and Highway Operations Research and Development (R&D) includes the Structures Division; Pavement Division; Construction, Maintenance, and Environmental Design Division; and the Materials Technology and Chemistry Division. The Office of Safety and Traffic Operations R&D includes the Systems Technology Division, Safety and Design Division, Traffic Control and Operations Division, and Urban Traffic Management Division. The reports are available from the address noted at the end of each description.

When ordering from the National Technical Information Service, use PB number and/or the report number with the report title.



Process Development for Production of Calcium Magnesium Acetate (CMA), Report No. FHWA/RD-82/145

by FHWA Materials Technology and Chemistry Division

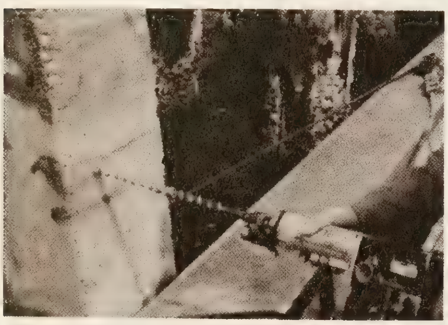
CMA is a candidate chemical for noncorrosive, environmentally acceptable, selective replacement of sodium chloride for highway and bridge deicing. This report discusses the alternative processes and feedstocks that were screened to identify a preferred process and feedstock for commercial production of CMA. The process identified as having the greatest economic potential is one in which the anaerobic, thermophilic bacterium, Clostridium Thermoaceticum, is used to ferment biomass-derived sugars to acetic acid; the acetic acid is then reacted with dolomitic lime to make CMA. Corn grain was the most favorable biomass feedstock in the tonnages projected as needed; high glucose corn syrup was a slightly less favorable choice.

Preliminary process designs were prepared, and detailed economic analyses were made for three alternative CMA processes—from corn grain by hydrolysis and fermentation, from high glucose syrup by fermentation, and from purchased synthetic acetic acid. Cost sensitivity analyses were included for the effects of plant size, feedstock costs, plant facilities investment, and plant onstream time. For the best case, the estimated selling price (plant gate) of CMA was about seven to eight times the current cost of road salt.

The experimental effort to develop the process to permit a pilot plant was hampered by delays in obtaining a seed culture of Clostridium Thermoaceticum and by subsequent indications that the seed culture received probably contained the spores of another organism. The report contains recommendations for further work needed to achieve the process performance assumed for the best case economic analysis.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.

Use of Improved Structural Materials Systems in Marine Piling, Report No. FHWA/RD-82/113

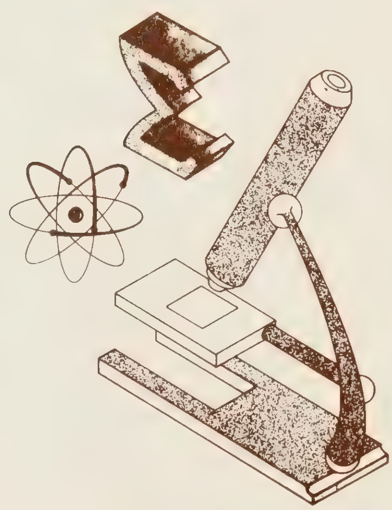


by FHWA Construction, Maintenance, and Environmental Design Division

This report contains the results of a study to evaluate the feasibility of manufacturing precast, prestressed marine piles from polymer concrete, polymer impregnated concrete, internally sealed concrete, and latex modified concrete. Included in the report are a description of the laboratory work that preceded the preparation of the specifications, a description of the manufacturing process and problems with each system, and the initial results of the short term performance of the various structural concretes.

Limited copies of the report are available from the Construction, Maintenance, and Environmental Design Division, HNR-30, Federal Highway Administration, Washington, D.C. 20590.

Performance of Highway Bridge Abutments on Spread Footings, Report No. FHWA/RD-81/184



by FHWA Construction, Maintenance, and Environmental Design Division

The use of spread footings to support highway bridge abutments and piers varies widely mostly because of a lack of confidence in their ability to perform adequately. To promote increased use and confidence, FHWA conducted a survey of existing bridges supported by spread footings on compacted fill. Results are discussed in this report.

The structural conditions of 148 highway bridges in the State of Washington were visually inspected. The approach pavements and other bridge appurtenances also were inspected for damage or distress that could be attributed to the use of spread footings on compacted fill.

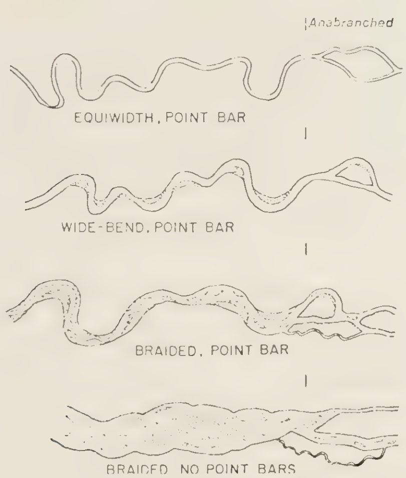
It was concluded that spread footings can provide a satisfactory alternative to piles especially when high embankments of good quality borrow materials are constructed over satisfactory foundation soils. None of the bridges investigated displayed any safety problems or serious functional distress; all bridges were in good or very good condition.

In addition to the performance evaluation, cost effectiveness analyses and tolerable movement correlation studies were made to substantiate further the feasibility of using spread footings instead of expensive deep foundation systems. Cost analyses showed spread footings were 50 to 65 percent less expensive than pile foundations.

Foundation movement studies showed that these bridges easily have tolerated differential settlements of 25–75 mm (1–3 in) without serious distress.

Limited copies of the report are available from the Construction, Maintenance, and Environmental Design Division, HNR-30, Federal Highway Administration, Washington, D.C. 20590.

Stream Channel Stability Assessment, Report No. FHWA/RD-82/021



by FHWA Structures Division

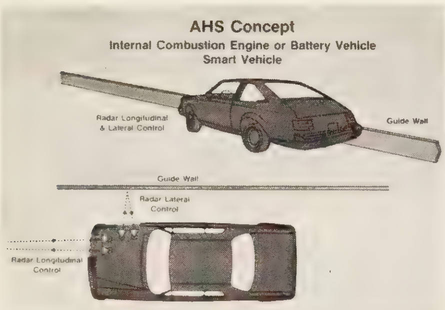
An unstable stream channel has a potential rate or magnitude of change great enough to be significant in the planning or maintenance of a bridge, highway, or other channel-related structure. Channel instability manifests itself as progressive lateral migration (bank erosion), progressive vertical change in bed elevation (degradation, aggradation), or fluctuation in bed elevation about an equilibrium value with changes in stream stage (scour and fill). Channel instability may cause hydraulic problems that result in structural damage to bridge abutment or pier foundations in, or adjacent to, the channel.

Channel instability is primarily a problem of alluvial streams. This report describes 14 major properties of alluvial stream channels on the basis of which channel stability can be evaluated. Some of the properties such as stream flow characteristics and the composition of the bed material may require field measurement for a complete understanding of channel behavior. However, lateral stability can be assessed reliably from an interpretation of other properties evident from aerial photographs taken at normal stream stage. These properties include the presence or absence of cut banks, the nature of point bars, and the variability of stream width.

Because certain combinations of stream channel properties tend to occur together, the report identifies four major alluvial stream types having different channel stability characteristics: Equiwidth, point bar; wide bend, point bar; braided, point bar; and braided, no point bar. Any of the four types may be anabranched locally or generally.

The results of measurements made on a select group of 36 streams indicate the rate and magnitude of lateral migration that may be expected for streams of different types and sizes. Some representative values for channel degradation and for scour also are given. Descriptions and illustrations of practical techniques for the engineering interpretation of stream channel stability from aerial photographs are presented.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 83 118190).



Systems Studies of Automated Highway Systems, Final Report, Report No. FHWA/RD-82/003

by FHWA Systems Technology Division

A comprehensive analysis of a system configuration and implementation costs and considerations for a potential future Automated Highway System has been completed. The Philadelphia metropolitan area is the background for the system configuration and the associated analysis of system use and cost.

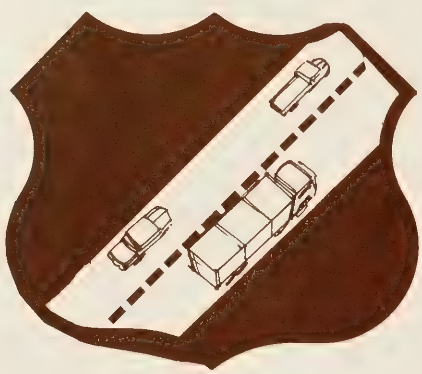
The Automated Highway System envisions private vehicles that can be operated fully automatically on a suitably equipped roadway or manually on conventional roads. Part of the study examined the extent of automated control instrumentation that should be contained in the vehicle versus the guideway for the most practical operation.

The study details the many advantages in increased capacity and improved safety resulting from an Automated Highway System as well as the predicted deployment costs.

In addition to the final report, the program documentation includes three appendixes—Appendix I: Conceptual Automated Highway System Design, Report No. FHWA/RD-82/129; Appendix II: Analysis of Automated Highway Systems, Report No. FHWA/RD-82/130; and Appendix III: Automated Highway System Trade Studies, Report No. FHWA/RD-82/131.

The four volumes are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Final Report—Stock No. PB 83 146266).

Passenger Car Equivalents for Rural Highways, Report No. FHWA/RD-82/132



by FHWA Traffic Control and Operations Division

This report describes a project to determine the passenger car equivalent value for different vehicle types under varying traffic and roadway geometric conditions. Field data collected in several States on both two- and four-lane highways were analyzed. Data included headways, speeds and travel times by vehicle type, traffic volume condition, and roadway section type. An analytical model was developed to estimate passenger car equivalent values based on speed distributions, traffic volume, and vehicle type. The calibrated model was used to estimate passenger car equivalent values for 14 vehicle types under the specified typical conditions for two- and four-lane rural highways.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.

Integrated Motorists Information System, Phase III: Development of Detailed Design and PS&E, Report No. FHWA/RD-82/108

by Urban Traffic Management Division

This report summarizes and describes the design and development of Plans, Specifications, and Estimates (PS&E) for the Integrated Motorist Information System (IMIS) being installed in the Northern Long Island Corridor in New York. The design of IMIS is documented by the following subsystems: surveillance, signing, ramp metering, arterial control, control center (including operator interface), central computer subsystem, remote sites, and communications. Operating procedures for freeway and arterial control, failure management, and incident management are summarized. Operational requirements for staffing, training, and equipment maintenance also are discussed.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.

Evaluation of Preferential Lanes for HOV's at Metered Ramps, Report No. FHWA/RD-82/121

by FHWA Traffic Control and Operations Division

The corridor selected for the study described in this report was the Golden State Freeway Corridor (I–5) between Route I–10 and State Route 170 in the Los Angeles area. Before, after, and control section data were analyzed to determine the effectiveness of the preferential bypass lanes.

On 13 of the 47 metered freeway onramps, bypass lanes were provided for buses and carpools with two or more occupants. The time saving was small and, although the number of carpools increased, the increase attributed to the meter bypass lanes alone is not considered significant. However, the bypass lane is a useful and effective means to support forming carpools and using transit modes for commuting.

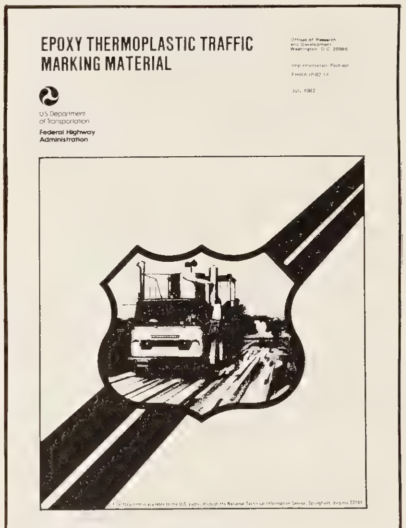
The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.

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

The following are brief descriptions of selected items that have been recently completed by State and Federal highway units in cooperation with the Office of Implementation, Offices of Research, Development, and Technology, Federal Highway Administration. Some items by others are included when they have a special interest to highway agencies.

U.S. Department of Transportation
Federal Highway Administration
Office of Implementation, HRT-1
Washington, D.C. 20590

Items available from the Office of Implementation can be obtained by including a self-addressed mailing label with the request.



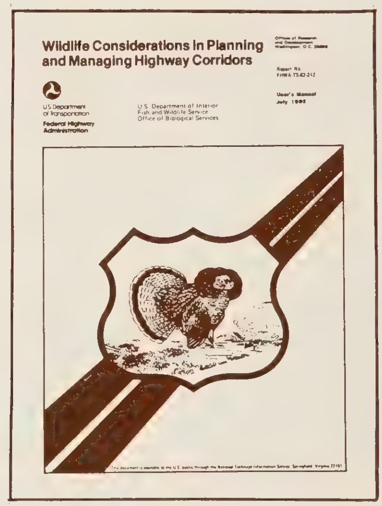
Epoxy Thermoplastic Traffic Marking Material, Report No. FHWA-IP-82-14

by Office of Implementation

This report summarizes the results of an effort to develop a more durable traffic marking material—Epoxy Thermoplastic (ETP). Background information on the development of ETP, a discussion of the field tests and evaluations, the material composition, and equipment modifications for applying ETP are included. The report also provides guidelines and specifications for purchasing and installing ETP and should be of interest to traffic and materials engineers concerned with roadway pavement markings.

Limited copies of the report are available from the Office of Implementation.

Wildlife Considerations in Planning and Managing Highway Corridors, Report No. FHWA-TS-82-212



by Office of Implementation

This report is a guide and information source for biologists, environmental specialists, and highway personnel concerned with route selection, design, construction,

operation, and maintenance of the Nation's highways. Highway-wildlife relationships and effects are discussed, information on inventorying wildlife populations is provided, and ways of incorporating wildlife values into highway planning are suggested, as are methods for managing wildlife populations within the highway right-of-way.

Limited copies of the report are available from the Office of Implementation.

Develop a Low Cost Raised Pavement Marker From EPOFLEX, Report No. FHWA/RD-82/502

by Office of Implementation

This report evaluates an inexpensive raised reflectorized marker that was developed from epoxy thermoplastic pavement marking material (EPOFLEX) to guide traffic through temporary construction zones under wet-night conditions. The research project included analysis and formulation modification, optimization of retroreflective materials, development of placement equipment, installation of field test sections, and evaluation.

Standard commercial EPOFLEX and highly modified EPOFLEX formulations were prepared and methods and equipment were developed to form inplace, round temporary markers appropriately reflectorized. Over 200 of these markers were installed in an actual construction zone. Methods were developed for removing the markers; after 3 months, the markers were removed.


DEVELOP A LOW COST RAISED PAVEMENT MARKER FROM EPOFLEX

Report No. FHWA/RD-82/502

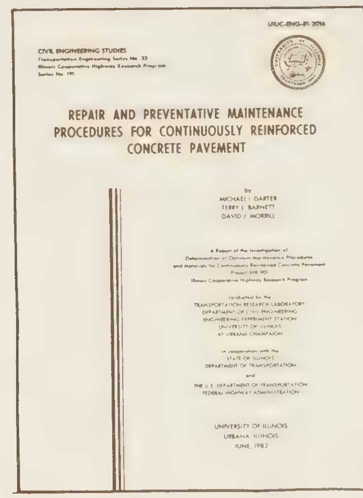
November 1982

U.S. Department of Transportation

Federal Highway Administration

It was found that none of the EPOFLEX formulations could hold their shape as installed. The heavy overriding of markers that occurs in construction zones mashed the EPOFLEX markers, destroying their capacity to perform as raised markers.

Limited copies of the report are available from the Office of Implementation.


CIVIL ENGINEERING STUDIES

REPAIR AND PREVENTATIVE MAINTENANCE PROCEDURES FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

UNIVERSITY OF ILLINOIS

URBANA, ILLINOIS

JUNE, 1982

Repair and Preventative Maintenance Procedures for Continuously Reinforced Concrete Pavement, Report No. FHWA/IL/UI-191

by the Illinois Department of Transportation and the Office of Implementation

Procedures for permanently patching and pressure grouting continuously reinforced concrete pavement have been developed and extensively field tested to insure their practicality and adequacy. This report presents these procedures and provides background information. The patching procedures reduce costs and lane closure time by considering the different distress types, different methods of construction, and concrete additive and curing for early opening. The pressure grouting procedures restore support beneath the slab and minimize future pumping.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 83 111427).

Computer Controlled Traffic Signal Systems, Report No. FHWA-IP-82-21

by Office of Implementation

This report provides technical and management procedures for developing an efficient traffic control signal system. It includes technical descriptions of the operation of signal system hardware and software and discussions of analytical techniques and management procedures required for feasibility studies, system design, and operations and maintenance of computer controlled traffic signal systems. The report is intended for traffic engineering personnel familiar with signal system operation.

Limited copies of the report are available from the Office of Implementation.

Pavement Patching Guidelines, Report No. FHWA-TS-82-221

by Office of Implementation

This report presents guidelines and recommendations for constructing patches during cold weather on both an emergency and routine basis and during warm weather on a routine basis. Patching of flexible, rigid, and composite pavements is discussed. Recommended patching techniques include using bituminous and portland cement patching materials. Pavement distress and causes are indexed to the recommended repair procedures.

Limited copies of the report are available from the Office of Implementation.

New Research in Progress

The following items identify new research studies that have been reported by FHWA's Offices of Research, Development, and Technology. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway or Transportation Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1W: Measurement and Evaluation of Pavement Surface Characteristics

Title: Data Collection Procedure for Use With Skid Resistance Measurement. (FCP No. 31W2034)

Objective: Consider factors that cause seasonal and short term variation in skid resistance. Develop uniform procedures for collecting, storing, processing, and applying these data.
Performing Organization: Nittany Engineers & Management, State College, Pa. 16801
Expected Completion Date: May 1984
Estimated Cost: $130,885 (FHWA Administrative Contract)

FCP Project 1X: Highway Safety Program Effectiveness Evaluation

Title: State Highway Safety Information Systems—State of Utah. (FCP No. 31X1026)

Objective: Provide financial and technical support to the State of Utah in linking the traffic, accident, and roadway inventory files to a common reference system.
Performing Organization: Utah Department of Transportation, Salt Lake City, Utah 84119
Expected Completion Date: November 1984
Estimated Cost: $110,460 (FHWA Administrative Contract)

FCP Category 2—Reduce Congestion and Improve Energy Efficiency

FCP Project 2Q: Exploiting New Technology to Improve Performance and Reduce Costs of Urban Signal Systems

Title: Functional Specifications for Microcomputer Based Traffic Signal Systems. (FCP No. 32Q1062)

Objective: Develop a functional catalog and specifications for microcomputer based multilevel distributed logic traffic signal systems using the latest technology.
Performing Organization: Sigmatek, Inc., Miami, Fla. 33166
Expected Completion Date: April 1984
Estimated Cost: $93,890 (FHWA Administrative Contract)

FCP Category 3—Environmental Considerations in Highway Design, Location, Construction, and Operation

FCP Project 3E: Reduction of Environmental Hazards to Water Resources Due to Highway Systems

Title: Effect of Bridge Repainting Operations on the Environment. (FCP No. 43E4113)
Objective: Determine the environmental toxicity for existing paint systems and evaluate toxicity of proposed paint systems. Evaluate existing paint/abrasive system residue for disposal suitability and existing cleaning materials and abrasives for environmental toxicity. Develop a testing procedure to evaluate future materials and guidelines for assessing potential environmental toxicity of repainting operations.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95819
Expected Completion Date: September 1985
Estimated Cost: $156,400 (HP&R)

FCP Category 3F: Pollution Reduction and Environmental Enhancement

Title: Revegetation of Highway Slopes in High Desert With Native Plant Seedlings. (FCP No. 43F1152)
Objective: Evaluate costs and compare success of revegetation of high desert slopes with seedlings from seeds locally collected and grown, outside area seeds locally grown, and seeds both collected and grown elsewhere.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: June 1985
Estimated Cost: $83,200 (HP&R)

Title: Microscale Air Quality Modeling and Mitigation in Mountainous Terrain. (FCP No. 43F4552)
Objective: Evaluate the effectiveness of implemented air quality mitigation measures by comparing with air quality measurements made before 1981-1982 road widenings, left turn lanes, and traffic response signal installations. Test validity of Caline 3 air quality model for use in complex rolling mountain terrain with one- to three-story structures nearby. Develop criteria for use of air quality models in moutainous terrain.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: June 1984
Estimated Cost: $70,500 (HP&R)

Title: Prevention of Traffic Noise Problems. (FCP No. 33F4653)
Objective: Develop a technical reference and acoustic procedures implementation package on ambient noise prediction and control that can be used by local jurisdictions to review plans and specifications submitted by a developer in highway traffic noise impacted areas.
Performing Organization: U.S. Department of Commerce, Washington, D.C. 20234
Expected Completion Date: December 1983
Estimated Cost: $124,840 (FHWA Administrative Contract)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4D: Remedial Treatment of Soil Materials for Earth Structures and Foundations

Title: Engineering Properties and Uses of Kentucky Shales. (FCP No. 44D5222)
Objective: Develop or identify inexpensive geotechnical testing procedures for characterizing shales to facilitate highway embankment and cut slope design.
Performing Organization: Kentucky Department of Transportation, Frankfort, Ky. 40506
Expected Completion Date: September 1984
Estimated Cost: $221,000 (HP&R)

FCP Project 4G: Substitute and Improved Materials to Reduce Effects of Energy Problems on Highways

Title: Asphalt Composition Analysis Using High Performance Liquid Chromatography (HPLC). (FCP No. 44G1543)
Objective: Devise a standard test procedure for determining the molecular size distribution of asphalt cement using HPLC.
Performing Organization: Colorado Department of Highways, Denver, Colo. 80222
Expected Completion Date: May 1984
Estimated Cost: $20,000 (HP&R)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5K: New Bridge Design Concepts

Title: Column Loads Due to Superstructure Shortening. (FCP No. 45K2062)
Objective: Study the structural response of reinforced concrete bridge columns through their service load and deformation histories as well as throughout their elastic, cracking, inelastic, and ultimate ranges.
Performing Organization: University of California, Berkeley, Calif. 94720
Funding Agency: California Department of Transportation
Expected Completion Date: October 1984
Estimated Cost: $152,000 (HP&R)

FCP Project 5L: Safe Life Design for Bridges

Title: Capacities of Bolted Joints for Bridge Timber Falsework. (FCP No. 45L3232)
Objective: Conduct 48 full-scale tests to determine the capacity of bolted joints of the kind used with falsework bents. Incorporate results of the study into the California Standard Specifications and the California Falsework Manual.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: December 1984
Estimated Cost: $40,000 (HP&R)

FCP Project 5N: Pavement Management Strategies

Title: The Use of Fuzzy Sets Mathematics to Assist Pavement Evaluation and Management. (FCP No. 45N2522)
Objective: Use fuzzy sets mathematics to combine scientific knowledge objective data with experience and engineering judgment subjective information to assist pavement evaluation and management.
Performing Organization: Purdue University, West Lafayette, Ind. 47907
Funding Agency: Indiana Department of Highways
Expected Completion Date: June 1985
Estimated Cost: $60,000 (HP&R)

FCP Category 6—Improved Technology for Highway Construction

FCP Project 6C: Use of Waste Materials for Highways

Title: Selection and Use of Fly Ash for Highway Concrete. (FCP No. 46C2312)
Objective: Develop a list of fly ash producing power plants located within practical shipping distance and capable of sending sufficient tonnages of the material to Indiana. Procure properly selected and authenticated samples of fly ash from as many of the major listed producers as practical. Determine and interpret the physical and chemical properties of the fly ashes obtained as they may affect the behavior of concrete in highway pavements.
Performing Organization: Purdue University, West Lafayette, Ind. 47907
Funding Agency: Indiana Department of Highways
Expected Completion Date: September 1984
Estimated Cost: $93,700 (HP&R)

Title: Evaluation of Hydraulic Cement Concretes Containing Slag. (FCP No. 46C2323)
Objective: Evaluate the effect of ground, granulated slags on the properties of concrete for highway use, workability, air void system, time of set, compressive strength, permeability, freeze-thaw resistance, drying shrinkage, heat of hydration, nature of carbonation, and hydration reactions. Establish a suitable nonproprietary specification for slag entailing an evaluation of proposed ASTM specifications.
Performing Organization: Virginia Department of Highways and Transportation, Richmond, Va. 23219
Expected Completion Date: June 1985
Estimated Cost: $104,800 (HP&R)

FCP Project 6D: Structural Rehabilitation of Pavement Systems

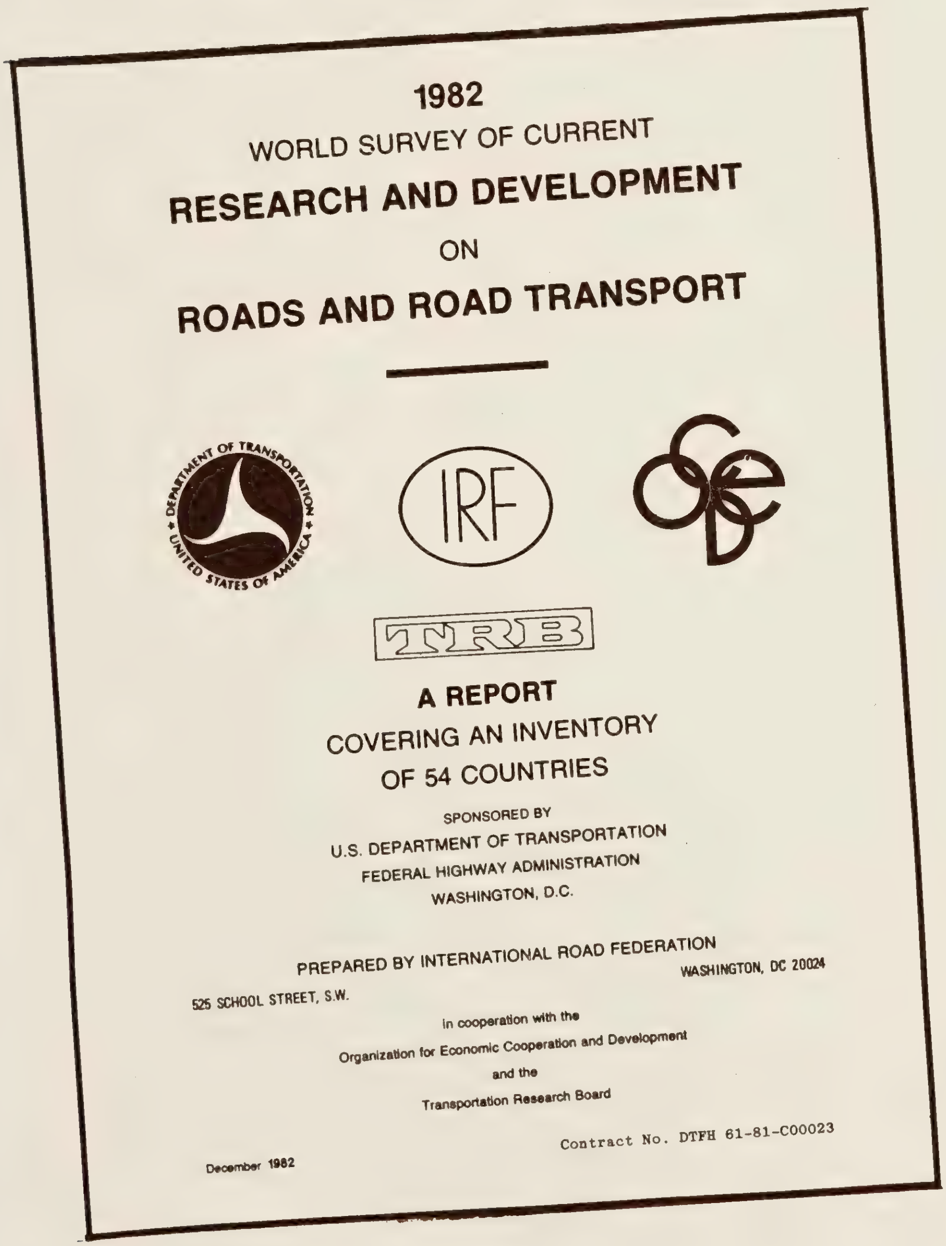
Title: Optimum Pavement Rehabilitation Scheduling Based on Extent of Load-Associated Cracking. (FCP No. 46D1502)
Objective: Perform field and laboratory testing and analysis on several pavement sections that have been monitored in the past and are known to have provided traffic service several years beyond the end of their design lives. Reexamine the 10 percent cracked level as an indicator of rehabilitation needs.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95807
Expected Completion Date: June 1984
Estimated Cost: $35,000 (HP&R)

FCP Category 0—Other New Studies

Title: Evaluation of Experimental Test Sections. (FCP No. 40M3013)
Objective: Validate the use of high performance liquid chromatography for asphalt analysis to predict serviceability of pavements.
Performing Organization: Montana State University, Bozeman, Mont. 59717
Funding Agency: Montana Department of Highways
Expected Completion Date: September 1988
Estimated Cost: $63,000 (HP&R)

Title: Computational Package for Pile Stress and Capacity. (FCP No. 40S8442)
Objective: Develop a computational package of design procedures for pile foundations. Predict pile stresses ahead of hard driving, thus minimizing potential construction problems.
Performing Organization: Purdue University, West Lafayette, Ind. 47907
Funding Agency: Indiana Department of Highways
Expected Completion Date: July 1985
Estimated Cost: $30,050 (HP&R)

New Publications



1982

WORLD SURVEY OF CURRENT

RESEARCH AND DEVELOPMENT

ON

ROADS AND ROAD TRANSPORT

A REPORT

COVERING AN INVENTORY

OF 54 COUNTRIES

SPONSORED BY

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

WASHINGTON, D.C.

PREPARED BY INTERNATIONAL ROAD FEDERATION

525 SCHOOL STREET, S.W. WASHINGTON, DC 20024

In cooperation with the

Organization for Economic Cooperation and Development

and the

Transportation Research Board

December 1982 Contract No. DTFH 61-81-C00023

The 1982 edition of the International Road Federation's (IRF) **World Survey of Current Research and Development on Roads and Road Transport** has been published. This report, the only worldwide published inventory of highway research in progress, is a cooperative effort of FHWA, the Transportation Research Board, and the IRF. It incorporates all ongoing research from the International Road Research Documentation of the Organization for Economic Cooperation and Development Program, which sponsors inventories in 17 countries, and also includes information from other countries.

Reports from Brazil, Japan, South Africa, and the United Kingdom identifying areas of perceived research need and the extent and nature of planned research are included as an appendix to the 1982 volume.

The 1983 edition of **World Survey of Current Research and Development on Roads and Road Transport** (available in January 1984) will identify and group research under a major heading in the main body of the report to make it easier to obtain information on specific areas. In addition, research activities in specific countries will be referenced in the index. This change will improve the content, form, and usefulness of the World Survey.

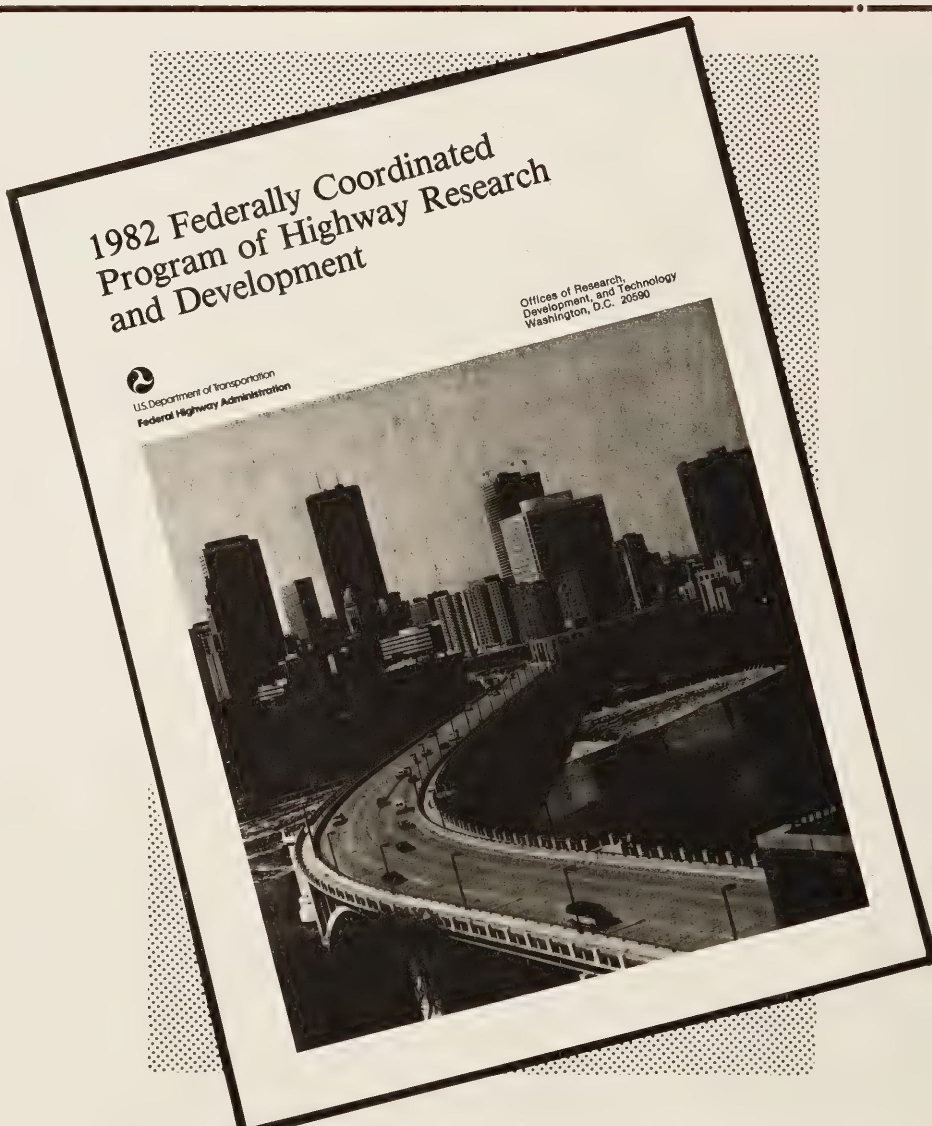
The annual World Surveys compile and record existing research projects in an effort to make known work being done in every country to avoid unnecessary duplication of effort and provide knowledge needed to overcome current problems. The goal is to assist countries in developing more productive highway research programs, which will mean better and safer roads, lower maintenance costs, and better standards of living.

Limited copies of the 1982 edition of **World Survey of Current Research and Development on Roads and Road Transport** are available from the International Road Federation, 525 School Street, SW., Washington, D.C. 20024. Comments and recommended format and content changes for future editions also should be sent to this address.

FHWA's Offices of Research, Development, and Technology (RD&T) have released their fiscal year **1982 Annual Report on the Federally Coordinated Program (FCP) of Highway Research and Development.**

This 40-page report briefly describes the goals and programs of the FCP and FCP accomplishments in highway research and development during FY 1982. Specific accomplishments in safety, traffic operations, the environment, materials, structures, highway construction, and highway maintenance are cited. Also highlighted are the major features of the recently reorganized Offices of RD&T and progress made in completing the expanded research facility in McLean, Va. The report is prefaced by a message from Federal Highway Administrator R. A. Barnhart.

While supplies last, individual copies of the report are available without charge from the Federal Highway Administration, Offices of RD&T, Office of Operations Staff, HRD-10, Washington, D.C. 20590.



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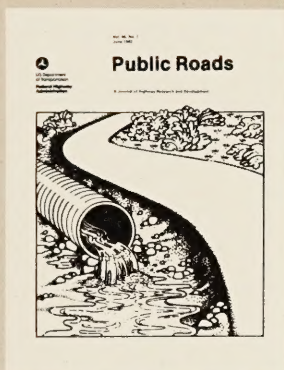

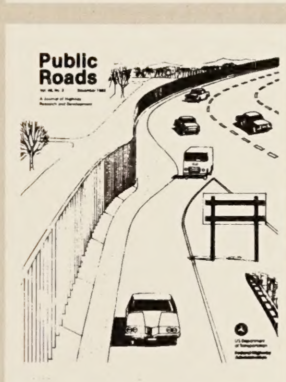

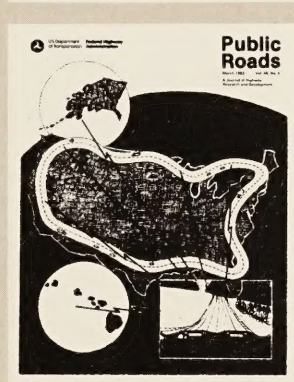

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