December 1991
Vol. 55, No. 3

U.S. Department of Transportation
Federal Highway Administration

Public Roads

A Journal of Highway Research and Development

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Public Roads, A Journal of Highway Research and Development (ISSN 0033-3735; USPS 516-690) is published quarterly by the Office of Research and Development, Federal Highway Administration (FHWA), 400 Seventh Street, SW, Washington, DC 20590. Second class postage paid at Washington, DC, and additional mailing offices.

POSTMASTER: Send address changes to *Public Roads, A Journal of Highway Research and Development,* HRD-10, FHWA, 6300 Georgetown Pike, McLean, VA 22101-2296.

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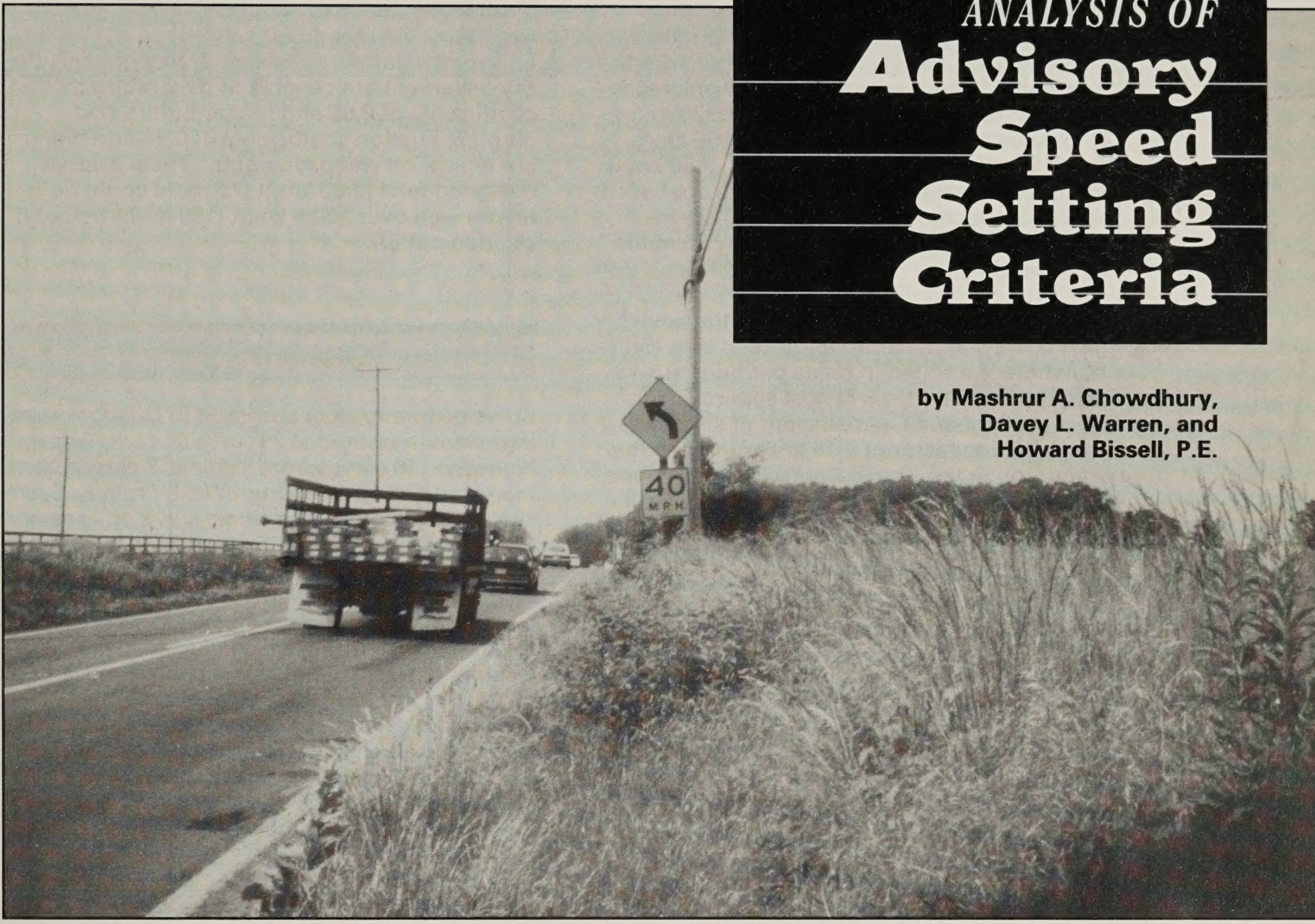
The Secretary of Transportation has determined that the publication of this periodical is necessary in the transaction of the public business required by law of this Department.

ANALYSIS OF

Advisory Speed Setting Criteria

by Mashrur A. Chowdhury, Davey L. Warren, and Howard Bissell, P.E.



Introduction

Curve warning signs are often supplemented with an advisory speed plate when the safe curve speed is less than the speed limit. The advisory speed is intended to inform an unfamiliar driver of a possible hazardous situation and recommend a safe speed to drive around the curve. Recent studies and surveys suggest that advisory speeds on curves are generally set too low and are not set consistently from State-to-State or even within a given State.*(1,2)* As a result the average motorist has little or no respect for the advisory speeds. Drivers using the highway repeatedly become accustomed to the speed that road condition and curvature will allow; consequently, the advisory speed signs do not have much affect on them. However, an unfamiliar motorist who finds it safe to drive 16.1 km/h (10 mi/h) over the advisory speed will be placed in a potentially hazardous situation when encountering the occasional curve posted with a realistic and meaningful advisory speed.

Despite dramatic improvements in tires and vehicle handling characteristics, current criteria for setting advisory speeds have remained essentially unchanged for over 50 years. The study described in this article evaluated the validity of current curve speed criteria for modern vehicles.

Background

Historically, advisory speeds for curves been determined in the field by making several trial runs through the curve at different speeds in a vehicle equipped with a ball-bank indicator. The ball-bank reading is a combined measure of

[1]Italic numbers in parentheses identify references on page 71.

centrifugal force, vehicle roll, and superelevation; as such, it indicates overturning forces on the vehicle. The generally accepted criteria for setting advisory speeds are ball-bank readings of 14° for speeds below 32 km/h (20 mi/h), 12° for speeds between 32 km/h and 56 km/h (20 and 35 mi/h), and 10° for speeds of 56 km/h (35 mi/h) or greater.*(3)* These criteria are based on tests conducted in the 1930's and are intended to represent the 85th to 90th percentile curve speed.*(4)*

Another method used to determine the advisory speed is the standard curve formula: $V^2 = 15R(e + f)$, where V = speed in miles per hour, R = radius of curve in feet, e = rate of superelevation in feet per foot and f = coefficient of side friction. A friction factor of 0.16 is assumed in the nomograph in the *Traffic Control Devices Handbook* (TCDH). A side friction factor of 0.16 corresponds to the speed at which discomfort begins for an average rider in a 1930 vintage car; this factor may not be valid for modern vehicles.*(5)* The side friction factors recommended in the design criteria of the American Association of State Highway and Transportation Officials (AASHTO) vary from 0.17 at low speed to 0.10 at the highest speed.*(6)* However, these values are also based on tests conducted in the 1930's and represent the limit at which a rider will notice a "side pitch" and begin to feel some discomfort. The ball-bank readings of 14°, 12° and 10° were found in these earlier studies to correspond to side friction values of 0.21, 0.18, and 0.15, respectively.

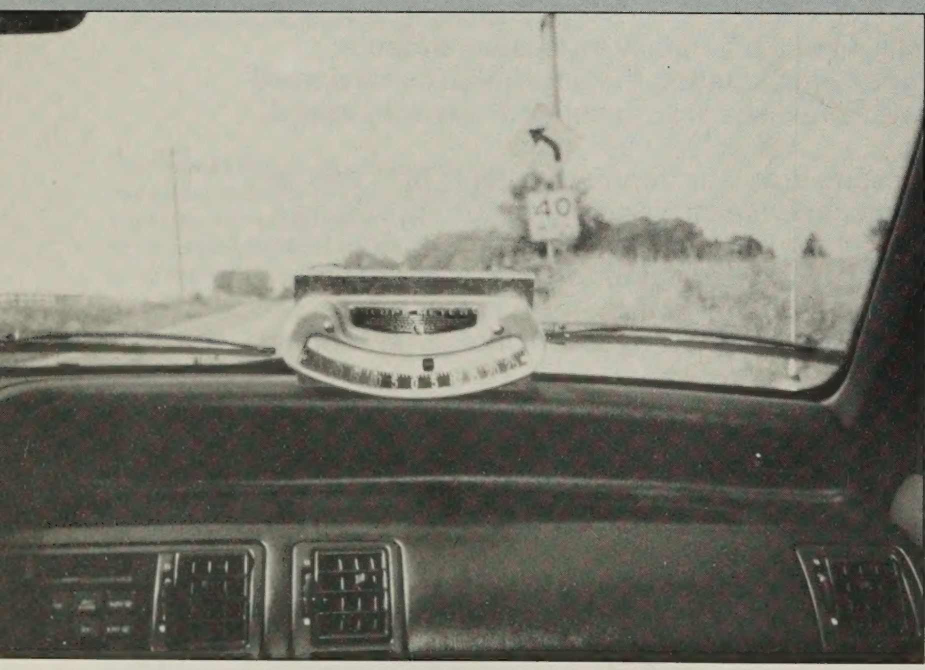


Figure 1.—Ball-bank indicator mounted on the windshield.

The friction factors used in current criteria do not reflect the maximum safe speed but rather an average comfortable speed. Modern cars on dry pavement are capable of generating friction coefficients of 0.65 and higher before skidding.*(5)* Friction coefficients of 0.40 and higher are typical on wet pavements. Thus, a curve designed for 112.63 km/h (70 mi/h) could be driven well over 160.9 km/h (100 mi/h) before it skidded out.*(6)*

Data Collection

Curve geometry, spot speeds, and ball-bank readings were measured at 28 curves on two-lane highways (16 curves from Virginia, 7 curves from Maryland and 5 curves from West Virginia). Each site consisted of either a single curve or, in case of a series of curves, the first curve. A few of the sites contained intersecting roads or driveways nearby, but volumes were low and did not have an appreciable impact on the observed speed of traffic. Field studies were conducted in dry weather between 9 a.m. and 4 p.m. on weekdays.

A pair of *radar speed readings* were taken on 50 free-flowing vehicles at each site. The data collector was positioned to measure the vehicle speeds running at the farthest visible point on the tangent approaching the curve and again at the middle of the curve.

Several *ball-bank readings* were taken at the sites by running tests with a vehicle (Chevrolet Celebrity station wagon, model 1984). The possibility of an inaccurate speedometer during test runs was eliminated by using a moving radar, which displayed the test vehicle speed while negotiating the curve. Before the test run at each site, the ball-bank indicator was mounted on the windshield of the test vehicle and adjusted on a level surface to bring the ball in the indicator to zero (see figure 1). Extra care was taken to drive the car parallel to the center line of the curve so as to avoid any bias in the ball-bank reading from cutting the curve short. The first trial run was made at the posted advisory speed. At least three additional trials were run in 8-km/h (5-mi/h) increments and the corresponding ball-bank readings were recorded. Each curve was also driven three times at a speed judged reasonable to the driver of the

Table 1.—Geometric and operating speed of the test sites

Site	Advisory Speed (mi/h)	Degree of Curve	Super-elevation	Operating Speed			
				Tangent		Curve Speed	
				50th	85th	50th	85th
C1VA	20	21	0.03	43	47	37	41
C2VA	25	50	0.10	38	42	29	32
C3VA	15	29	0.10	37	42	32	36
C4VA	25	25	0.13	45	50	40	43
C5VA	15	16	0.04	36	41	30	35
C6VA	25	29	0.03	43	49	32	35
C7VA	45	9	0.09	58	64	55	58
C8VA	50	8	0.07	52	56	50	55
C9VA	50	7	0.07	54	59	51	55
C10VA	40	7	0.04	52	56	51	55
C11VA	40	10	0.06	51	56	46	51
C12VA	35	7	0.09	53	59	50	56
C13VA	40	8	0.10	50	54	48	53
C14VA	35	5	0.09	55	59	55	59
C15VA	25	13	0.03	41	44	37	42
C16VA	20	19	0.10	46	53	36	42
C17MD	40	8	0.05	53	60	49	54
C18MD	25	59	0.10	47	55	26	30
C19MD	25	38	0.07	53	57	32	35
C20MD	25	12	0.04	47	53	42	46
C21MD	25	28	0.09	45	51	32	38
C22MD	25	27	0.09	41	45	37	41
C23MD	30	16	0.03	37	42	33	37
C24WV	35	21	0.10	44	47	38	41
C25WV	35	18	0.13	46	50	41	45
C26WV	45	10	0.08	49	56	44	49
C27WV	35	19	0.10	49	56	41	45
C28WV	50	9	0.05	51	58	50	55

1 mi/h = 1.609 km/h

Table 2.—Percent compliance with advisory speeds

Advisory Speed, (mi/h)	Percent Compliance	Range (percent)
15-20	0	
25-30	8	0-38
35-40	5	0-32
45-50	43	0-68

1 mi/h = 1.609 km/h

test vehicle; and the corresponding ball-bank readings were recorded.

At each site, the *degree of curve* was determined by measuring the offset in inches from edgeline to the midpoint of a 18.9-m (62-ft) chord. Superelevation was measured along the curve with a carpenter's level. Each of these measurements was taken three times and the average recorded. Table 1 shows the geometric and operating speed of the test sites.

Motorist Compliance

On average, 9 out of 10 motorists exceeded posted advisory speeds. Moreover, zero compliance with posted advisory speeds was observed at almost half of sites. Table 2 shows the percentage of motorist compliance with posted speeds at each site.

Motorists generally exceeded the advisory speeds posted below 48 km/h (30 mi/h) by wider margins than they did for advisory speeds posted at 56 km/h (35 mi/h) or higher. The per-

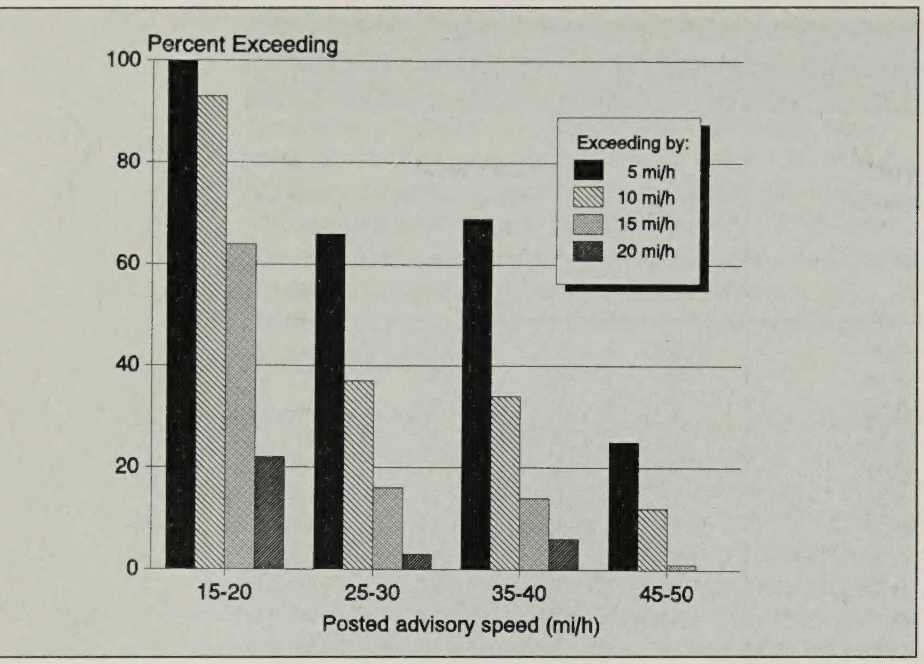


Figure 2.—Percent exceeding posted advisory speeds by various amounts.

centage of drivers exceeding the advisory speeds by various ranges are shown in figure 2.

Although compliance with the advisory speeds was poor, drivers *did* adjust their speeds on curves. Table 3 compares the speed reduction suggested by the posted advisory speed and the actual speed drop. The *suggested speed reduction* is the difference between the observed speed on the approach and the posted advisory speed. The *actual speed drop* is the difference between the approach speed and curve speed. Only the average speed reductions observed in West Virginia were near the suggested value. Overall, drivers were advised to slow down an average of 24.1 km/h (15 mi/h); however, the actual speed drop averaged only 9.6 km/h (6 mi/h). The advisory speed at the test sites was thus exceeded on average by 14.5 km/h (9 mi/h).

Posted Speeds, Recommended Speeds and Observed Speeds

To determine whether highway agencies are employing the ball-bank indicator or the TCDH nomograph in setting the safe curve speeds, the actual recommended speed at each site was compared to the speeds derived via these methods.

Table 4 shows the recommended speed of each curve based on the ball-bank indicator criteria and nomograph in the TCDH, which is based on the standard curve speed formula with a friction factor of 0.16. Only West Virginia actually posted advisory speeds consistent with generally recommended criteria. In Maryland, about one-half of the curves were posted according to existing criteria, and most were within 8 km/h (5 mi/h). Less than one-third of the curves in Virginia corresponded to the recommended speed based on the standard ball-bank criteria or side friction value of 0.16; many were set 16 km/h (10 mi/h) or more below the recommended value.

Table 3.—Average speed reduction on curves

State	Speed Drop in mi/h	
	Suggested	Actual
Virginia	15.8	4.6
Maryland	18.7	10.4
West Virginia	7.9	4.9
All Curves	15.1	6.1

1 mi/h = 1.609 km/h

Table 5 compares differences in the posted and recommended speeds relative to 50th- and 85-percentile curve speed. The recommended speed based on the nomograph better reflects driver behavior than that based on the ball-bank indicator. Even so, the recommended speed would still be below the average speed of traffic. Table 6 shows the sites arranged from the sharpest to the flattest curve, superelevation, friction demanded, and ball-bank readings corresponding to the 85th percentile speed.

Ball-Bank Reading Corresponding to Observed Speeds

The fact that the recommended speeds based on existing ball-bank criteria are below the prevailing speed of traffic suggests that the current criteria are not valid for modern vehicles. The ball- bank readings corresponding to the 50th- and 85th-percentile curve speeds are plotted in figure 3. The line represents the best fit linear regression. Although higher ball-bank readings were observed at lower speed curves, the ball-bank values do not conform well with existing criteria. The 85th-percentile ball-bank readings are some 5 to 10° higher than existing criteria.

On average, the 50th- and 85th-percentile speeds corresponded to ball-bank readings of 14 and 17°. However, there is a great deal of scatter in the data; this casts doubt on using the ball-bank indicator to establish safe curve speed even with revised readings.

Table 4.—Posted advisory speed and recommended speed

Site	Posted Speed (mi/h)	Recommended Speed		
		Ball-Bank	Nomograph	85th Percentile
C1VA	20	30	30	41
C2VA	25	25	25	32
C3VA	15	25	30	36
C4VA	25	30	30	43
C5VA	15	30	35	35
C6VA	25	30	25	35
C7VA	45	45	50	58
C8VA	50	55	50	55
C9VA	50	50	50	55
C10VA	40	45	50	55
C11VA	40	45	45	51
C12VA	35	55	55	56
C13VA	40	55	50	53
C14VA	35	60	60	59
C15VA	25	30	35	42
C16VA	20	30	35	42
C17MD	40	40	45	54
C18MD	25	20	25	30
C19MD	25	25	25	35
C20MD	25	35	35	46
C21MD	25	30	30	38
C22MD	25	30	30	41
C23MD	30	30	35	37
C24WVA	35	35	35	41
C25WVA	35	35	35	45
C26WVA	45	45	45	49
C27WVA	35	30	35	45
C28WVA	50	50	50	55

1 mi/h = 1.609 km/h

Table 5.—Difference in posted advisory speed and recommended speed relative to observed speed

	50th Percentile	85th Percentile Speed
Posted Advisory Speed	-8.8	-13.0
Ball-Bank	-3.6	-7.8
Nomograph	-2.4	-6.6

Side Friction

As mentioned previously, the TDCH nomograph for determining safe curve speed is based on the standard design speed formula using a side friction value of 0.16. Figure 4 shows the friction factor corresponding to the 50th and 85th percentile speeds of each curve computed from the standard curve speed formula in the TCDH and AASHTO design guide. The line in the figure represents best fit linear regression.

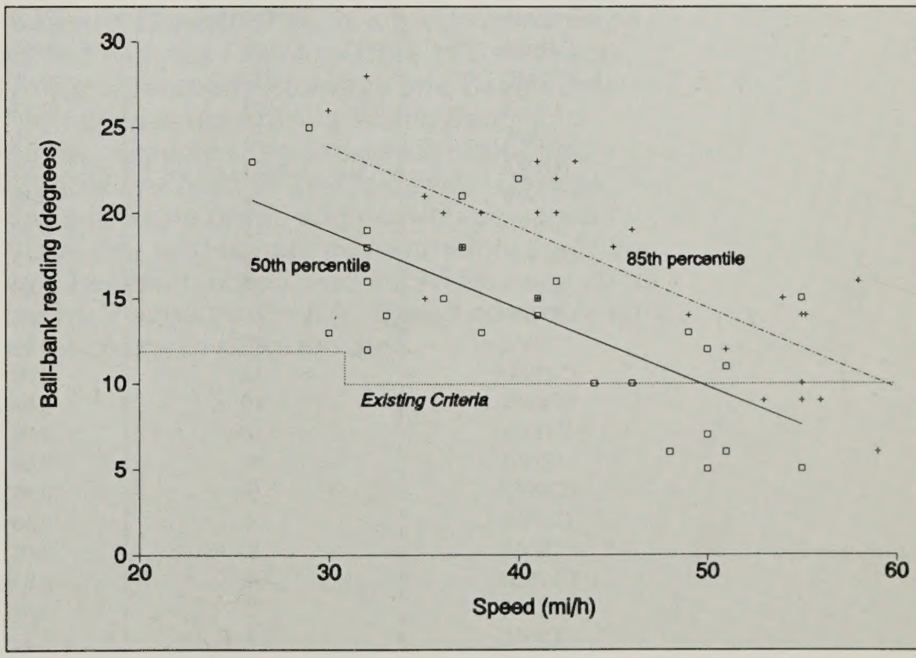


Figure 3.—Ball-bank readings corresponding to 50th- and 85th- percentile speeds.

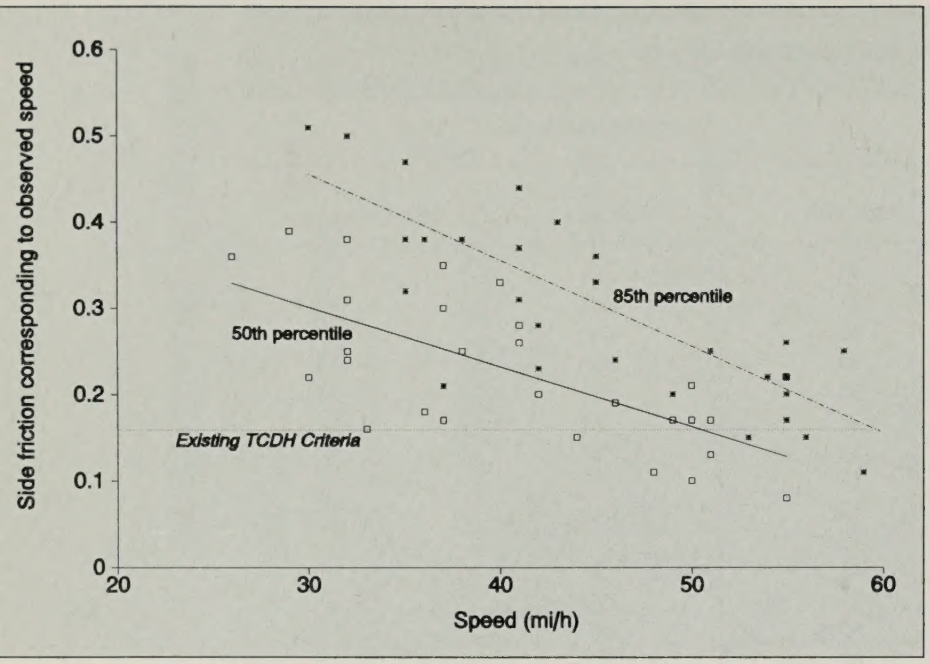


Figure 4.—Side friction corresponding to 50th- and 85th-percentile speeds.

The average friction used by 50th-percentile drivers was 0.22; it was 0.29 for 85th-percentile drivers. These values are nearly twice the current value used in establishing advisory curve speeds. Figure 4 shows drivers accept higher side friction on lower speed curves than is currently assumed for road design. However, the friction values used by drivers are at least 50 percent greater than those assumed in road design. For comparison, modern cars on dry pavements can generate side friction values ranging from 0.65 to 0.90 before skidding out.*(5)* Thus, the friction used on curves is limited more by driver comfort than by the limits of the vehicle-pavement interaction.

Conclusions

The absence of adequate and universally accepted criteria for determining advisory speeds creates the problem of nonuniform and subjec-

Table 6.—Friction demanded and ball-bank readings corresponding to 85th percentile speeds

Site	Degree of Curve	Super-elevation	85th Percentile Speed	Friction From 85th Percentile Speed	Ball-Bank Readings from 85th Percentile Speed
C18MD	59	0.10	30	0.52	26
C2VA	50	0.10	32	0.5	28
C19MD	38	0.07	35	0.47	21
C3VA	29	0.10	36	0.34	20
C6VA	29	0.03	35	0.38	15
C21MD	28	0.09	38	0.38	20
C22MD	27	0.09	41	0.44	23
C4VA	25	0.13	43	0.41	23
C24WV	21	0.10	41	0.31	15
C1VA	21	0.03	41	0.38	20
C27WV	19	0.10	45	0.35	18
C16VA	19	0.10	42	0.29	22
C25WV	18	0.13	45	0.29	18
C5VA	16	0.04	35	0.19	15
C23MD	16	0.03	37	0.22	18
C15VA	13	0.03	42	0.24	20
C20MD	12	0.04	46	0.26	19
C26WV	10	0.08	49	0.20	14
C11VA	10	0.06	51	0.24	12
C7VA	9	0.09	58	0.26	16
C28WV	9	0.05	55	0.27	14
C13VA	8	0.10	53	0.16	9
C8VA	8	0.07	55	0.21	9
C17MD	8	0.05	54	0.22	15
C12VA	7	0.09	56	0.17	9
C9VA	7	0.07	55	0.18	10
C10VA	7	0.04	55	0.21	14
C14VA	5	0.09	59	0.11	6

1 mi/h = 1.609 km/h

tive applications; this problem in turn poses a potential safety threat to unfamiliar drivers. The posted advisory speeds have little significance for the motorists. At most curves, posted advisory speeds were well below the prevailing traffic speed.

The study also found noticeable variation in the application of the existing ball-bank criteria from curve to curve and State to State. In most cases, the TCDH ball-bank criteria result in very low and unrealistic advisory speeds. The current criteria of 10, 12 and 14° should be revised upwards. Ball-bank readings of 12° above 64 km/h (40 mi/h), 16° between 48 km/h (30 mi/h) and 64 km/h (40 mi/h), and 20° below 48 km/h (30 mi/h) would better reflect observed or average curve speeds.

The nomograph and design speed formula are marginally better than the ball-bank indicator in reflecting driver speed behavior on curves (see table 6), but the current values used for side friction are too conservative. Although not very reliable (as it varied widely for a given speed), friction values of 0.30 at lower speed and 0.20 at higher speed would provide a more realistic determination of safe curve speed.

An alternative approach to determining safe curve speed would be to sample vehicular speeds. A sample of 10 vehicles could be used to estimate the average curve speed to within ±5 km/m (3 mi/h). This approach is currently being investigated as are several other alternatives for recommending safe speeds on curves. These alternatives include prediction models of curve speed based on degree and length of curve and use of the G-analyst, an accelerometer that provides a direct measure of lateral acceleration.

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FATIGUE TESTING FOR THERMEX REINFORCING BARS

by Yash Paul Virmani, William Wright, and Ronald N. Nelson

Introduction

Conventional grade 60 reinforcing steel that meets the American Association of State Highway and Transportation Officials (AASHTO) M31-89 (ASTM A615-87) specifications is generally manufactured by alloying certain percentages of rare metals, such as manganese and vanadium, with steel ingots to obtain the required strength. These rare metals are available mostly from foreign sources; 64 percent of the known world reserve of vanadium ore is in South Africa.

Because of the unavailability of these rare metals locally and their cost, the Florida Steel Corporation in 1987 decided to produce grade 60 reinforcing bars by a proprietary Thermex process which reduces the requirements of these alloying elements. This process, developed in eastern Europe, is similar to the "Tempcore" process that has been in use in western Europe since 1974.

Thermex processing combines conventional hot-rolling with a water quenching and a self-tempering heat treatment. The reinforcing bar's exterior is cooled rapidly to a low temperature, facilitating the formation of a harder outer layer that approaches a martensitic condition, while the rebar interior is of lower strength and still quite ductile.

Although the Thermex process produces reinforcing steel that meets the AASHTO M31-89 specifications for grade 60 reinforcing steel, detailed research showed that Thermex steel has different physical properties from conventional reinforcing steel containing alloying elements. (*1*) Consequently, before it would permit large-scale use of reinforcing steel in Federal-aid projects, the Federal Highway Administration (FHWA) investigated the fatigue characteristics of Thermex versus conventional reinforcing steel. The findings of this investigation are described in this article.

Background

Currently, the AASHTO material specification for reinforcing steel (M31-89 grade 60) has no provision for requiring rebars to meet certain fatigue requirements. Moreover, there is no standard test method—or agreement on best method—of performing fatigue test on rebars in the U.S. To develop the AASHTO design specifications, testing was performed on rebars encased in concrete beams loaded in flexture (2). This process simulates the conditions experienced by rebars in service but is impractical for quality control testing of the bars. More recent work has been performed by the firm Wiss, Janney, Elstner, Associates, Inc. (WJE), where rebars were tested in air in a servo-hydraulic load frame. (*3*) The WJE method correlated fairly well with the concrete encased tests, however, the air test results were very sensitive to the method used for gripping specimens.

Unlike the United States, the British do have a material standard that requires rebars to meet specified fatigue performance criteria. (*4*) This standard reflects work performed at the Transport and Road Research Laboratory (TRRL) in the United Kingdom, in which specimens were axially tested in air. (*5*) The British specification requires rebars to survive 5 million load cycles at a prescribed stress range that is dependent on the diameter of the bar being tested. Based

[1]Italic numbers in parentheses identify references on page 77.

on the experience at TRRL and WJE, it was decided to test bars in air for the present study.

Description of Specimens

To resolve the fatigue question, the FHWA requested that the Florida Department of Transportation supply appropriate samples of both Thermex and conventional reinforcing steel to perform fatigue tests at the Turner-Fairbank Highway Research Center (TFHRC) laboratories.

Initially, the TFHRC received 24 rebar samples from the Florida Steel Corporation for testing. These consisted of four Thermex and four conventional bars in three different bar sizes—#5, #6, and #11. The bars were all 460 mm (18 in) long. The Thermex bars were designated by the letter "T" on the bar. These 24 original specimens were used in resolving the "grip problem" discussed below. At a later date, additional samples of #5 and #11 rebars were shipped by the Florida Steel Corporation mill in Knoxville, Tennessee, for further testing.

The manufacturer provided chemical analysis data for both the Thermex and conventional bars. The FHWA obtained a chemical analysis on the same bars by an independent testing laboratory. Table 1 shows the chemical analysis data for both bar types. All residual elements were excluded from the table because they have little effect on tensile properties.

Comparing the Thermex to the conventional bars showed no significant difference in manganese content; however, the carbon content was about 35 percent lower in the #5 Thermex bars than in the conventional bars. The concentration of vanadium, which is only added to conventional bars larger than the #6 size, was much lower in the Thermex than the conventional #11 bars. The percent of vanadium was negligible in the #5 bars produced by both processes. This last finding substantiates the manufacturer's claim that the addition of vanadium is not required in Thermex- produced bars larger than #6 in order to obtain the tensile strength required by the AASHTO M31-89 grade 60 specification.

Fatigue Testing

All fatigue tests were performed on bars mounted axially in a servo-hydraulic load frame. Cyclic loading was applied at a rate of 20 Hz for the #5 and #6 bars; however, this had to be reduced to 10 Hz for the #11 bars because of the higher loads required. These frequencies resulted in rapid testing of the specimens, with most tests lasting less than 24 hours.

Initially, the rebars were tested at a stress range of 138 MPa (20 ksi), with the minimum to maximum stress ratio of R = 0.2. However, based on discussion with researchers from the United Kingdom's Transport and Research Laboratory, the test stress ranges used in Britain were later adopted, resulting in stress ranges of 200, 185, 150 MPa (29, 26.8, and 21.8 ksi) for the #5, #6, and #11 bars, respectively. (*4*) Tests were also conducted at a stress ratio of R = 0.2.

Table 1.—Chemical analysis of Thermex and conventional rebar samples

Process size	Bar size	Source	C (%)	Mn (%)	V (%)
Thermex	#5	Manufacturer	0.28	0.70	0.002
		Independent	0.29	0.63	<0.005
	#11	Manufacturer	0.31	0.82	0.003
		Independent	0.41	0.97	<0.005
Conventional	#5	Manufacturer	0.43	0.71	0.002
		Independent	0.44	0.75	<0.005
	#11	Manufacturer	0.34	1.01	0.028
		Independent	0.36	0.99	0.033

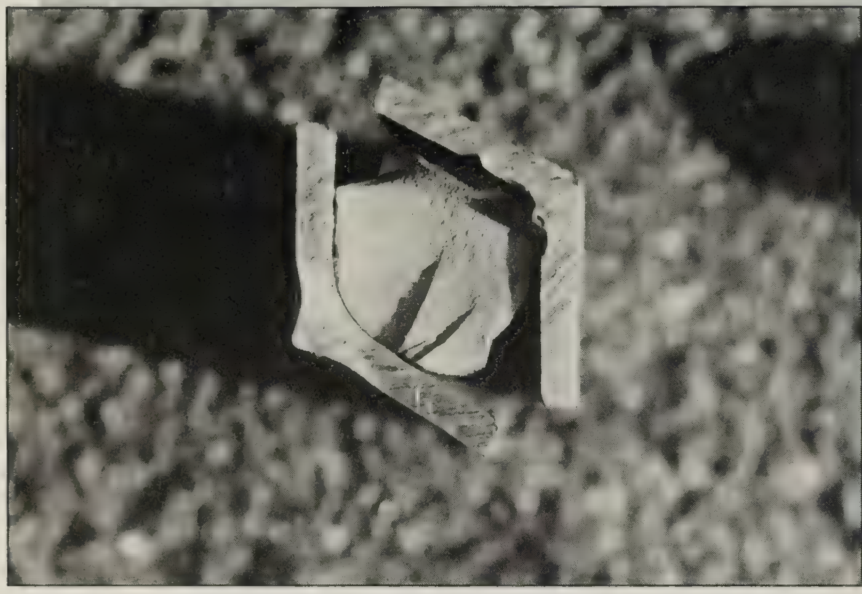

Figure 1.—Top view of a typical grip failure with aluminum shims.

Gripping Methods

Previous research has shown that axial tests of bars in air are very sensitive to the way they are gripped in the test machine. (*3,5*) To eliminate the gripping effects from the test results, several methods of gripping specimens were investigated. Many difficulties arise when trying to test reinforcing bars in a servo-hydraulic load frame. Unlike standard test specimens, the bar diameter cannot be reduced in the center section before testing. The deformations rolled onto the bar surface have an effect on fatigue life, and must remain on the bar during testing. Additionally, the Thermex process produces a hardened surface layer that also must remain. Simply put, the grip problem is that the bar has the same section inside and outside the grip area. Any force exerted by the grips will result in locally higher stress concentrations—which in turn will result in a high probability of failure at the grips.

Special wedge grips filled with babbitt (a lead-based alloy containing 1 to 10 percent tin and 10 to 15 percent antimony) were used for rebar fatigue testing conducted in the 1960's. More recently, in the 1988 National Cooperative Highway Research Project study, *Fatigue Behavior of Welded and Mechanical Splices in Reinforcing Steel,* WJE developed a special hollowed grip that used a high early strength grout to hold the bar. (*3*) During fatigue testing, the grout "clamps" the bar deformations in the grip by direct bond between the grout and bar. WJE had a high success rate of fatigue failures away from the grips in this National Cooperative Highway Research Program (NCHRP) pilot test program.

The current FHWA test program tried to find a simplified method of bar gripping, thereby speeding up the test procedure. Four types of grip systems were investigated at the TFHRC to try to get failure to occur away from the grip area. The following are brief descriptions of these grip systems:

1. *Aluminum or brass shims placed between the bar and the self-aligning, V-type wedge grips.* These shims deformed around the bar's raised deformations and prevented the wedge grips from marring bar surface. This method was ineffective in preventing grip failure. Figure 1 shows a top view of a typical grip failure with aluminum shims.

2. *Machining raised deformations off the bar ends, then inserting bars in the wedge grips lined with aluminum shims.* It was hoped that eliminating the stress concentration effects caused by the raised deformations would prevent grip failure. This method was effective for conventional type reinforcement; however, grip failure was still evident with the Thermex bars.

3. *Embedding bar ends in soft (babbitt) metal contained within conical grips.* The bar was placed in the body of a standard 12.7-mm (0.5-in) prestressing chuck, and molten metal was

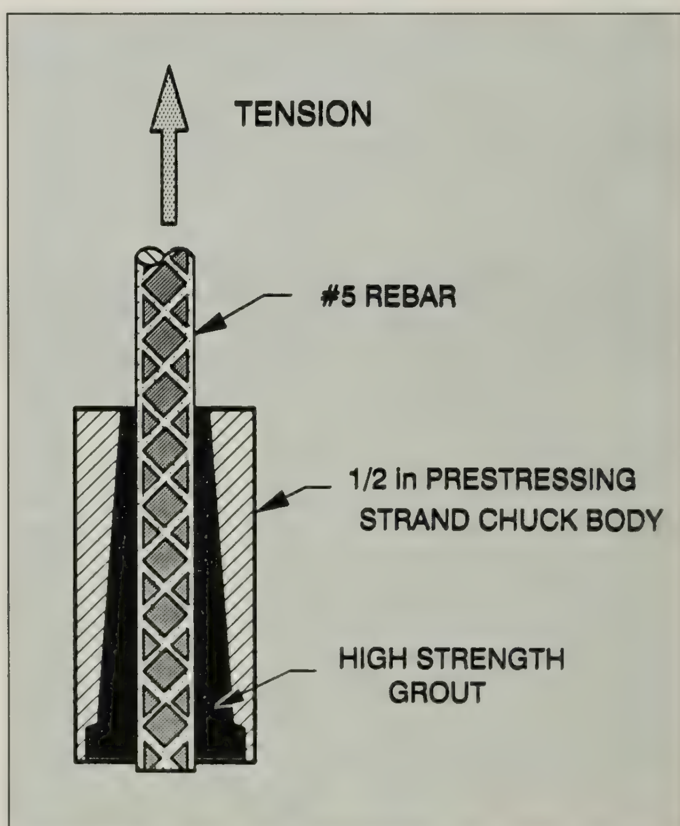

Figure 2.—Sectional view of grouted conical grip used for testing #5 bars.

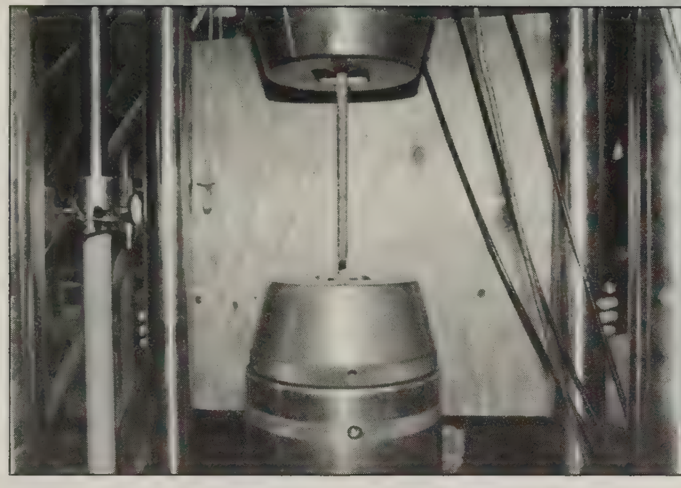

Figure 3.—Hydraulic load frame showing grouted conical grips mounted in self-aligning wedge grips.

poured around the bar. The prestressing chuck was then gripped in the self-aligning grips of the hydraulic load frame. Figure 2 shows a sectional view of this grip system. Both pure lead and babbitt metal were tried in this system. The lead proved to be too soft and fatigued before the bar; the babbitt metal was ineffective in preventing grip failure.

4. *Embedding bar ends in high strength metallic grout contained within the same conical grips.* This system is essentially the same as the WJE grout system mentioned above. Out of the four systems tested, the grouted conical grips were the most effective in preventing grip failure. Figures 3 and 4 show a typical failure occurring about 1 bar diameter outside of a grouted grip. The size of the prestressing chuck body limited testing with this system to #5 and smaller bars. Larger, specially made grips would be required for larger bars.

Findings

Grips

Table 2 shows the results of all the fatigue tests performed, indicating the grip type and location of failure. The failures noted as occurring outside the grip area (i.e., at least 1 bar diameter away from the grips) were considered valid.

For the conventional bars, the machined-end method was very effective, with valid failures occurring in 75 percent of the tests. For the Thermex bars, the grout and babbitt embedding methods were the only ones providing valid breaks. Of these two methods, the grout appeared to be the most effective, with both tests showing valid failures. The grout method should also work well for the conventional bars, although no tests were performed for verification.

It is apparent that the gripping medium is critical to the validity of tests conducted on reinforcing bars. In service, while some of the axial force is transferred to the rebar by adhesion, most of the axial force is transmitted through direct bearing between the concrete and the raised deformations along the bar. This circumstance results in a shear force at the point where the raised deformation joins the bar's round section. The magnitude of this force is determined by the compressive strength of the grout or other gripping medium. If the shear force introduced by the grip system is too great, large reductions in fatigue strength will result. This effect seems more critical in Thermex than conventional bars, probably because of the hardened surface layer on the Thermex bars.

Figure 5 shows the results of all the tests performed at the TFHRC, with different symbols indicating the types of grips used. The filled symbols indicate "valid breaks"— i.e., the bar broke at least 1 bar diameter outside the grip area. The machined-end and shim-grip systems seem to produce failures earlier than the grout- or babbitt-grip systems. Although data are limited, the grout system seems to produce a higher percentage of valid failures than the babbitt system. For this reason, grouted wedge-

Figure 4.—Closeup of a typical failure in the grouted conical grip.

Table 2.—Fatigue test data for Thermex and conventional rebar tests

Bar size	Steel type[a]	Grip type[b]	Stress range (ksi)	Cycles to failure	Failure location
#5	T	Machined	**29**	359,685	Upper grip
	T	Machined	**29**	292,304	Upper grip
	T	Machined	**29**	463,451	Upper grip
	T	Machined	**29**	489,815	Upper grip
	T	Babbitt	**29**	818,088	Upper grip
	T	Babbitt	**29**	1,514,763	1 diameter above lower grip[c]
	T	Babbitt		668,087	Lower grip
	T	Grout			1 diameter above lower grip[c]
	T	Grout			1 diameter above lower grip[c]
	C	Machined	**29**	505,267	Away from grips[c]
	C	Machined	**29**	616,621	Away from grips[c]
	C	Machined	**29**	533,742	Away from grips[c]
	C	Machined	**29**	391,772	Lower grip
#6	T	Shims	**20**	1,531,979	Lower grip
	T	Shims	**20**	1,410,895	Lower grip
	T	Shims	**20**	933,857	Upper grip
	T	Shims	**26.8**	569,762	Lower grip
	T	Shims	**26.8**	580,895	Lower grip
	C	Shims	**20**	1,690,682	Upper grip
	C	Shims	**20**	1,993,692	Upper grip
	C	Shims	**20**	6,441,999	No failgure, terminated
	C	Machined	**21.4**	524,792	Away from grips[c]
	C	Machined	**21.4**	482,556	Away from grips[c]
#11	T	Machined	**21.8**	757,995	Lower grip
	T	Machined	**21.8**	510,811	Upper grip
	C	Machined	**21.8**	1,161,825	Away from grips[c]
	C	Machined	**21.8**	658,629	Upper grip

[a]T = Thermex Bars; C = Conventional Bars

[b]Machined = raised deformations machined off the ends of the bar, gripped in wedges with brass shims; Shims = gripped in wedges using brass or aluminum shims; Babbitt = bedded in conical grips with babbitt metal; and Grout = bedded in conical grips with high strength grout.

[c]Break was outside of grip area.

Ksi * 6,895 = MPa

type grips should probably be specified if a standard test method for reinforcing steel is developed, even though they are more labor intensive than the other types investigated.

Thermex Versus Conventional Bars

Statistical findings

Figure 6 shows the fatigue test results of the three conventional and three Thermex #5 bars shown in table 2 that are considered valid. These tests all failed at points at least 1 bar diameter outside the grip area. As a reference, the solid and dotted lines in the figure represent, respectively, the lower 95-percent confidence limit and mean regression lines for #5 bars. (*4*) The Thermex bars all fell *above* the mean line while the conventional bars fell slightly *below* the mean line. All of the bars failed well above the lower 95-percent confidence limit.

Given the small sample size and the fact that fatigue tests usually show a large degree of scatter, no statistical conclusions can be drawn from these data. However, the data do tend to

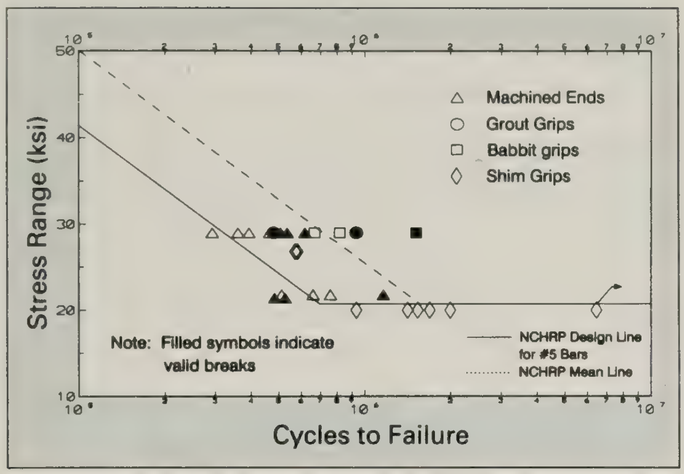

Figure 5.—Fatigue results showing the effect of grip type.

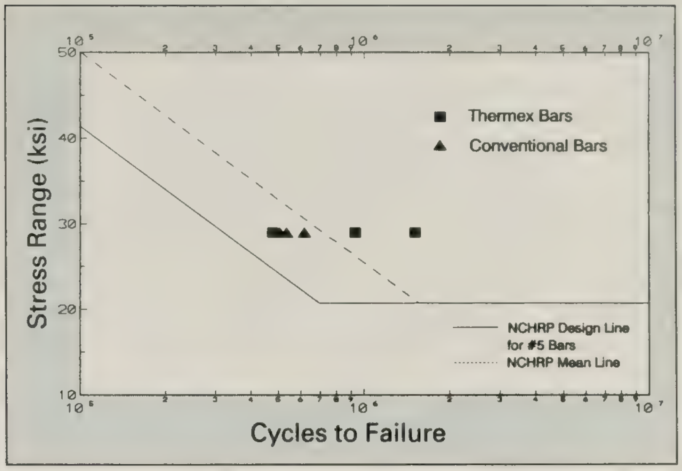

Figure 6.—Fatigue results for #5 bars comparing the Thermex and conventional processes.

support conclusions from testing performed by WJE —i.e., that there does not appear to be any significant difference in fatigue resistance between Thermex and conventional bars. (*6*)

Comparison with British standard

It should be noted that there is a large discrepancy between these test results and the British standard. For example, for #5 bars, the British standard requires testing at a stress range of 200 MPa (29 ksi). (*4*) The NCHRP mean line indicates shown in figure 6 indicates the average life of bars to be only about 700,000 cycles at 200 MPa (29 ksi) stress range. It could be argued that the NCHRP mean line was derived primarily from tests on bars embedded in reinforced beams loaded in flexure while the British standard was derived from tests of bars in air. The in-air tests performed by both WJE and the FHWA are relatively close to the NCHRP mean line, indicating that no significant difference exists between the in-air and reinforced-beam tests. Thus, the fatigue strength of U.S. bars seems relatively insensitive to test type. The large difference between the U.S. tests and the British standard remains unexplained.

Conclusions

This preliminary research developed insufficient data to make any conclusions regarding the relative fatigue resistance of bars produced by the Thermex process. It does, however, point up the need to develop a U.S. specification for testing reinforcement bars. What constitutes a valid test needs to be resolved and a level of acceptable performance specified. The discrepancy between U.S. test results and the British standard also should be investigated to see if U.S.-produced bars actually show lower fatigue resistance than those produced in Europe.

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(*5*) D.S. Moss. *Axial Fatigue of High Yield Reinforcing Bars in Air*, Supplementary Report 622, Transport and Road Research Laboratory, Crowthorne, Berkshire, United Kingdom, 1980.

(*6*) Letter from Conrad Paulson; Wiss, Janney, Elstner, Inc. Northbrook, to James Oliver, Florida Steel Corporation, Tampa, regarding fatigue tests of reinforcing bar, February 18, 1991.

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Laboratory Evaluation of Verglimit and PlusRide

by Kevin D. Stuart and Walaa S. Mogawer

Introduction

One technique to help control the formation of ice on asphalt pavements, and possibly reduce the use of salt, is to include deicing additives in the wearing course mixture. Two such additives are Verglimit and PlusRide rubber. Using these additives, however, increases pavement material and construction costs. These increases may be justified if ice-related accidents are reduced and the properties of the asphalt mixture are not adversely affected.

Background

Description of additives

This article describes a recent study, conducted by the Federal Highway Administration (FHWA) at its Turner-Fairbank Highway Research Center that investigated the effects of Verglimit and PlusRide rubber on the performances of asphalt mixtures. Specifically, the study examined the additives in terms of their effects on moisture damage, rutting, and low temperature cracking. The tests conducted included indirect tensile strengths, incremental creep moduli and permanent strains, and repeated load moduli and permanent strains. Specimens were also moisture conditioned to determine retained tensile strength ratios (TSR), retained resilient modulus ratios (MrR), and visual stripping. This study supplements various field trials now underway; most field studies formally evaluate only the effects on skid resistance and not other mixture or pavement properties.

Verglimit consists of 0.1- to 5-mm (0.004- to 0.21-in) particles of calcium chloride with a small amount of sodium hydroxide. The particles are encapsulated with linseed oil or polyvinyl acetate to seal the material and keep it inactive until the particles break under the action of traffic. The calcium chloride additive then mixes with moisture from the air or on the pavement to form a dilute salt solution. Verglimit is designed to work throughout the life of the pavement. Because it generally triples the cost of the bituminous mixture, Verglimit is used only in selected problem areas. The additional cost is not offset by the reduction in sanding and salting operations, but may be offset if it can be found that its use reduces accidents.

PlusRide is a patented bituminous mixture which contains granulated tire rubber. Most rubber particles are 0.16 to 0.64 cm (1/16 to 1/4 in). They act as elastic aggregates that flex on the pavement surface under traffic. This flexing helps to break up ice. PlusRide doubles the cost of the mixture and, like Verglimit, has been used only in selected problem areas.

Field applications

As noted above, several field studies are now under way to determine if and how Verglimit and PlusRide reduce the number of ice-related accidents. These studies are summarized below. Many of these field studies have indicated that Verglimit and PlusRide can affect mixture properties.

Verglimit

- A Verglimit project in New York performed well in light snowfalls, but was less effective in heavy snowfalls. No problems with pavement performance were observed in 8 years. (*1*)[1]
- Verglimit did melt ice in two Colorado projects, but the deicing action was so slow the effects were often masked by normal salting and sanding operations. (*2*) In an earlier Colorado project, the pavement raveled. This raveling was attributed to poor quality control at the hot-mix plant and during pavement construction. The pavement was also slick because of the attraction of a high amount of moisture to the surface. It was found that the Verglimit particles crushed by the roller quickly absorbed moisture from the air. Slickness on the other projects in Colorado was controlled by applying sand on the pavement after construction.
- In Pennsylvania, slickness was controlled by first sanding the surface and later flushing it several times with water. (*3*) This practice is now recommended by the supplier. The slick surface was attributed to the moisture absorption of the crushed Verglimit particles, although the linseed oil that encapsulates the calcium chloride was suggested as a contributing factor.
- No deicing benefits were found in Minnesota, and portions of a pavement shoved and were replaced. (*4*)

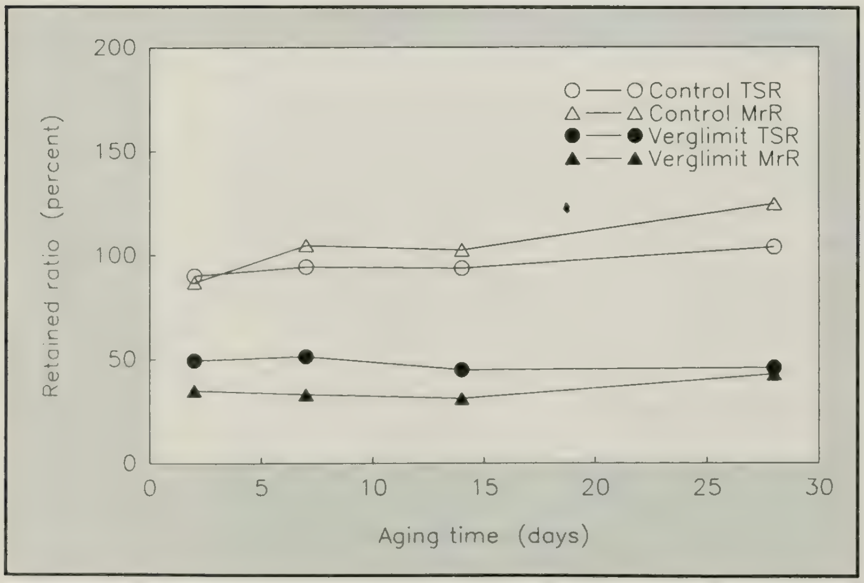

Figure 1.—Verglimit: MrR and TSR versus aging time.

[1]Italic numbers in parenthese identify references on page 86.

- In California, although poor compaction led to raveling problems, icing nevertheless decreased. (*5*)
- Some Verglimit sections in Oregon and New Jersey were replaced because of raveling. (*6*)

PlusRide

- The Alaska Department of Transportation installed experimental pavement sections using PlusRide in Fairbanks and Anchorage. (*7*) On measuring vehicle stopping distances, significant reductions in distance were observed during icy conditions as compared to control sections. Condition surveys were also made; these showed some raveling.
- The New Jersey Department of Transportation constructed a test site using PlusRide on route NJ41 in Cherry Hill. (*8*) Periodic skid tests showed that PlusRide improved the skid resistance of the pavement. Initial condition surveys indicated that there was slightly more rutting in the PlusRide section than in the control section. The rate of rutting then slowed, and the section subsequently performed acceptably.
- A PlusRide project was evaluated over 5 years by the State of Washington. (*9*) The control used was the State's standard asphalt-rubber open-graded asphalt concrete. The required density could not be obtained in the PlusRide section, and the air void level was close to 12 percent. Sections of the PlusRide material had to be patched. It was concluded that the PlusRide material did not give better frictional properties, noise reduction, or service life.
- Rhode Island constructed a 1.61-km (1-mi) test section composed of equal segments of PlusRide, Verglimit, and a control pavement. Since there were no differences in the performances of the three sections, the increases in cost due to the Verglimit and PlusRide were not offset.

Testing Program

A 50-blow Marshall mixture design was performed on each mixture to determine its optimum asphalt content. Specimens were then fabricated at this optimum content and tested for resistance to moisture damage, rutting, and low temperature cracking. The complete testing program is shown in table 1. All asphalt, aggregate, and mixture tests were performed accord-

Table 1. Testing program

Resistance to moisture damage at 25 °C (77 °F)

(Diametral tests on Marshall size specimens)

- Tensile strength ratio
- Resilient modulus ratio
- Visual percent stripping

Resistance to rutting

(10.16- by 20.32-cm {4- by 8-in} cylinders)

- Incremental creep modulus and permanent strain at 18.3, 25, and 40 °C (65, 77, and 104 °F)
- Dynamic modulus and permanent strain at 18.3, 25, and 40 °C (65, 77, and 104 °F)

Resistance to low temperature cracking

(Diametral tests on Marshall size specimens)

(Temperatures from -30 to 90 °F (-34.3 to 32.2 °C)

- Resilient modulus versus temperature
- Tensile strength versus temperature

ing to American Association of State Highway and Transportation Officials (AASHTO) and other recommended practices.

Specimens tested for their *resistance to moisture* damage were cured at 25 °C (77 °F) for 2, 7, 14, 28, or 90 days to determine if curing had an effect on adhesion and the deicers.

Low temperature effects were determined at a reference modulus of 20,700 MPa (3,000 ksi) and a reference tensile strength of 2.07 MPa (300 psi) in accordance with current practices.

The data described in this article are from four mixtures: PlusRide, PlusRide control, Verglimit, and Verglimit control. Additional mixtures were tested; the results from these tests support the data presented in this paper.

Resistance to Moisture Damage

Verglimit

Data from the moisture damage tests are given in table 2 and the TSR and MrR are presented in figure 1. Verglimit had a significant effect on the retained ratios, providing retained ratios lower than the control and also below suggested pass-fall criteria (70 percent for MrR and 80 percent for TSR). Verglimit thus increased the susceptibility to moisture damage. However, there was no visual stripping, and the low retained ratios were attributed to the high amount of swelling in the specimen that occurred during the 24-hour 60 °C (140 °F) water soak.

PlusRide

Data from the moisture damage tests are given in table 3 and TSR and MrR are presented in figure 2. PlusRide decreased both the TSR and MrR

Table 2. Moisture damage test results: Verglimit

Test	Control Days				Verglimit Days			
	2	7	14	28	2	7	14	28
Tensile strength (psi)								
Wet	65.8	68.9	68.4	75.8	38.6	40.3	35.0	35.8
Dry	73.0	72.1	90.0	78.3	78.2	104.2	81.7	59.9
Resilient modulus (ksi)								
Wet	73.1	88.0	86.0	104.6	45.0	42.5	39.8	54.8
Dry	84.0	113.2	155.2	133.8	128.4	184.4	134.8	76.8
Retained ratio (percent)								
TSR	90.1	95.6	76.0	96.8	49.4	38.7	42.8	59.8
MrR	87.0	77.7	55.4	78.2	35.0	23.0	29.5	71.3
Visual stripping (percent)								
	<5	<5	<5	<5	<5	<5	<5	<5

(ksi)(6895) = (KPa) (psi)(6895) = (Pa)

Table 3. Moisture damage test results: PlusRide

Test	Control					PlusRide				
	Days					Days				
	2	7	14	28	90	2	7	14	28	90
Tensile Strength (psi)										
Wet	74.8	81.2	73.1	67.9	96.9	55.6	55.3	47.0	43.4	55.4
Dry	74.5	89.3	87.8	72.4	98.7	60.5	60.4	62.7	50.9	60.0
Resilient Modulus (ksi)										
Wet	176.0	137.8	197.4	147.5	230.6	83.6	64.1	67.2	56.8	89.2
Dry	190.1	231.3	270.9	184.2	234.9	120.8	145.8	138.5	96.8	114.2
Retained Ratio (percent)										
TSR	100.4	90.0	83.3	93.8	98.2	91.9	91.6	75.0	85.3	92.3
MrR	92.6	59.6	72.9	80.1	98.2	69.2	44.0	48.5	58.7	78.1
Visual Stripping (percent)										
	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5

(ksi)(6895) = (KPa) (psi)(6895) = (Pa)

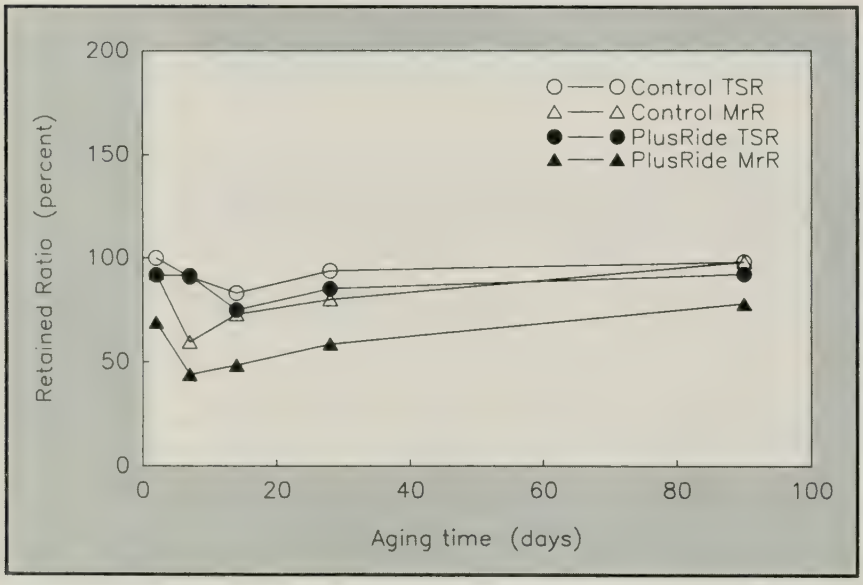

Figure 2.—PlusRide: MrR and TSR versus aging time.

and thus increased the susceptibility to moisture damage. No visual stripping was observed for this PlusRide mixture, but the specimens swelled during the water soak. Other PlusRide mixtures tested did, however, show some slightly higher levels of visual stripping compared to their control mixtures.

Resistance to Rutting

Verglimit

Creep moduli and permanent strains are presented in table 4. Verglimit increased the creep modulus and decreased the permanent strain at the high temperature for each creep time. Verglimit generally decreased the creep modulus but had a variable effect on permanent strain at low temperature. Overall, this indicated a slight decrease in the susceptibility to rutting.

The dynamic moduli at the 200th cycle and test temperatures of 18.3, 25, and 40 °C (65, 77, and 104 °F) for the control mixture were 5900, 3100, and 740 MPa (850,000, 450,000, and 107,000 psi), respectively. For the Verglimit mixture they were 4200, 3100, and 700 MPa (610,000, 443,000, and 102,000 psi). Verglimit caused a reduction in stiffness at the low temperature but had no effect at the other two temperatures. This finding generally indicates there should be no effect on the resistance to rutting.

Permanent strains versus the number of cycles at 18.3, 25, and 40 °C (65, 77, and 104 °F) are pre-

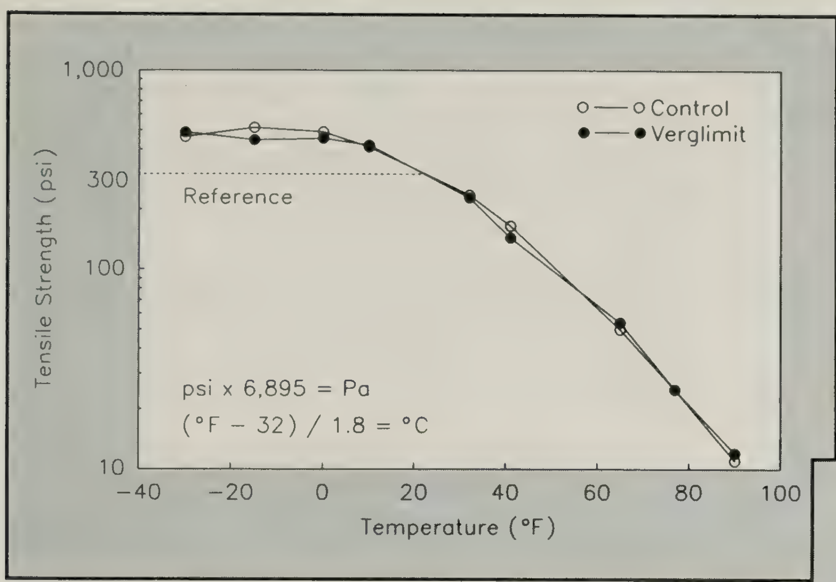

Figure 3.—Verglimit: tensile strength versus temperature for evaluating low temperature cracking.

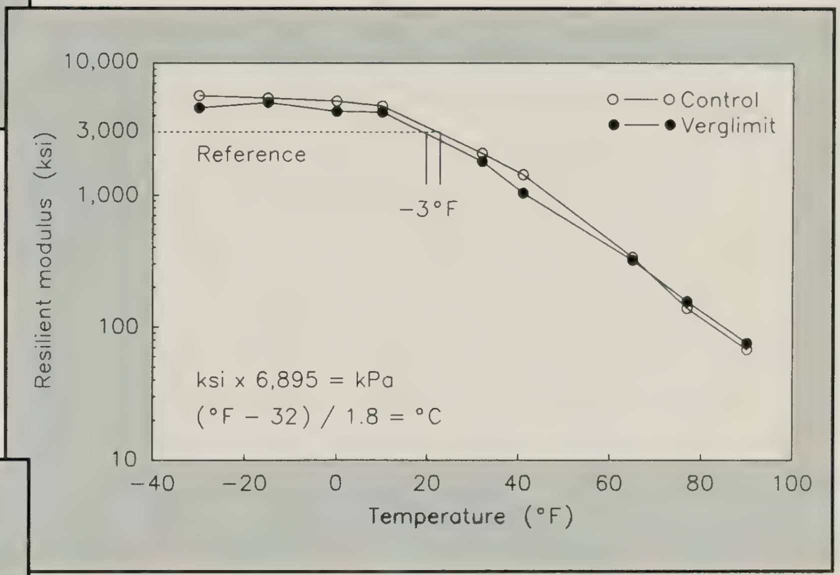

Figure 4.—Verglimit: resilient modulus versus temperature for evaluating low temperature cracking.

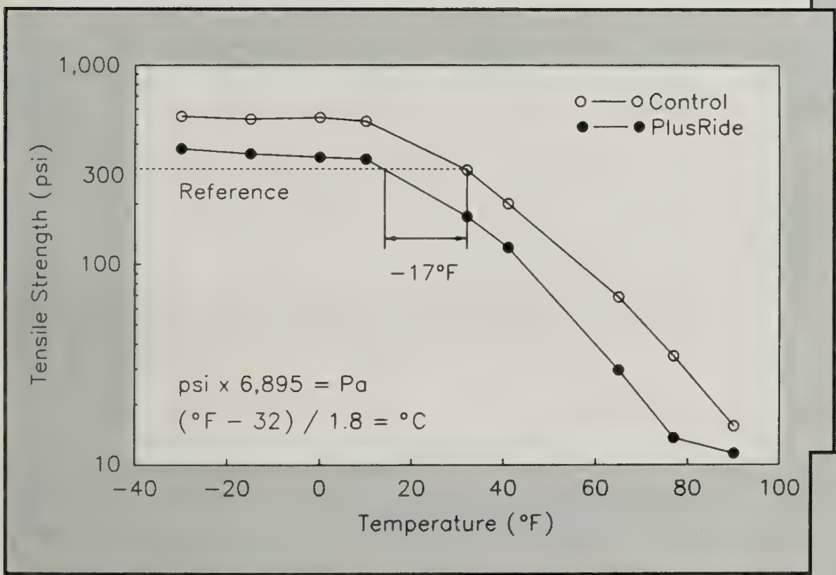

Figure 5.—PlusRide: tensile strength versus temperature for evaluating low temperature cracking.

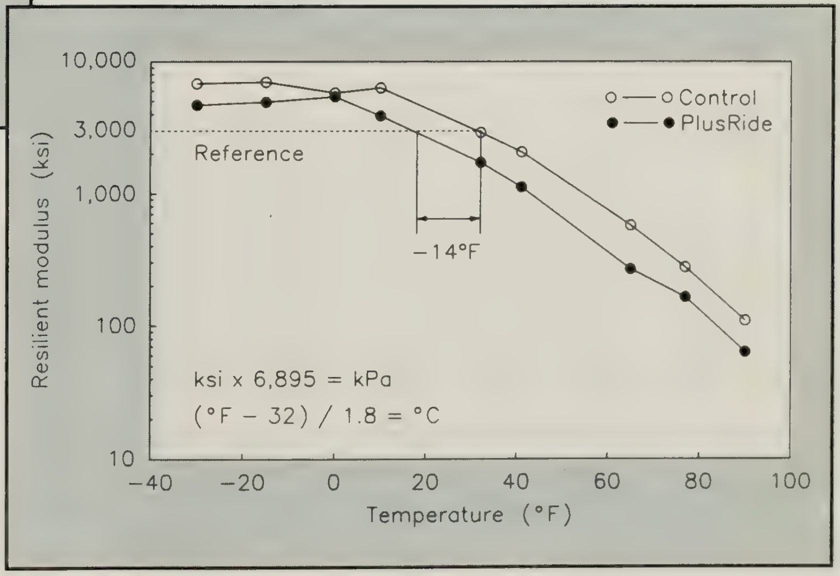

Figure 6.—PlusRide: resilient modulus versus temperature for evaluating low temperature cracking.

Table 4. Resistance to rutting—creep test results: Verglim

	Control			Verglimit		
Creep Time	Temperature			Temperature		
(sec)	65 °F	77 °F	104 °F	65 °F	77 °F	104 °F
	Creep modulus (ksi)			Creep modulus (ksi)		
0.10	508	254	62	278	242	94
0.30	273	146	49	178	147	68
1.0	145	85	44	107	99	58
3.0	78	72	45	74	74	53
10.0	66	60	43	56	59	49
30.0	53	48	—	51	56	45
100.0	46	42	—	49	51	38
1,000.0	38	32	—	42	47	27
	Permanent Strain (microinches)			Permanent Strain (microinches)		
0.10	28	70	281	12	43	34
0.30	72	161	493	63	129	245
1.0	148	274	688	131	207	365
3.0	240	325	811	282	324	454
10.0	346	422	871	419	464	445
30.0	389	437	—	516	375	422
100.0	560	653	—	550	608	783
1,000.0	833	1,144	—	759	800	1,684

(ksi)(6895) = (KPa) (in)(2.54)=(cm) (°F - 32)/1.8 = °C

Table 5. Resistance to rutting—repeated load test results: Verglimit

	Control			Verglimit		
number of	Temperature			Temperature		
cycles	65°F	77°F	104°F	65°F	77°F	104°F
	Permanent Strain (microinches)			Permanent Strain (microinches)		
1	64	174	443	29	73	203
3	111	329	709	62	151	339
10	191	618	1,035	141	295	552
30	328	940	1,581	239	465	1,081
100	400	1264	2,515	310	660	1,522
200	462	1,384	3,636	374	768	2,162
300	464	1,505	4,508	390	836	2,648
400	469	1,559	5,156	410	882	3,096
500	471	1,603	5,938	421	917	3,495
1,000	475	1,795	——	460	1,065	——
3,000	496	2,216	——	552	1,618	——
10,000	606	3,462	——	671	2,694	——
20,000	673	——	——	780	——	——
30,000	734	——	——	861	——	——
40,000	782	——	——	919	——	——
50,000	845	——	——	968	——	——

(in)(2.54)=(cm) (°F - 32)/1.8 = °C

sented in table 5. Verglimit reduced the amount of permanent strain at 25 and 40 °C (77 and 104 °F), and slightly increased the permanent strain at 18.3 °C (65 °F) at the high number of cycles. Thus, unlike the dynamic moduli, the permanent strains indicated a reduced susceptibility to rutting and a slight trend toward decreasing temperature susceptibility. Permanent strains are a much better indicator of performance than moduli, and thus the conclusions from the strain data should be used.

PlusRide

Creep moduli and permanent strains are presented in table 6. PlusRide decreased the creep modulus and increased the permanent strain at all temperatures and creep times. This indicated an increased susceptibility to rutting.

The dynamic moduli at the 200th cycle and test temperatures of 18.3, 25, and 40 °C (65, 77, and 104 °F) for the control mixtures were 7000, 3000, and 1100 MPa (1,010,000, 439,000, and 165,000 psi) respectively. For the PlusRide mixture they were 3200, 1500, and 500 MPa (467,000, 218,000, and 72,000 psi). PlusRide caused a reduction in modulus at all temperatures.

Permanent strains versus the number of cycles at 18.3, 25, and 40 °C (65, 77, and 104 °F) are presented in table 7. PlusRide increased the amount of permanent strain at all temperatures and cycles. Both the dynamic moduli and the permanent strains indicated an increased susceptibility to rutting.

Low Temperature Cracking

Verglimit

Tensile strengths and resilient moduli versus temperature are presented in figures 3 and 4, respectively. The resilient modulus test produced a shift of -0.6 °C (-3 °F) at the 20,700 MPa (3,000 ksi) reference modulus for the Verglimit mixture relative to the control. This shift was due to a slight decrease in temperature susceptibility. The tensile strength test at 2.07 MPa (300 psi) produced virtually equal data for both the Verglimit and control mixtures, and no shift was found. Overall, both mixtures would be expected to behave similarly in the field.

PlusRide

Tensile strengths and resilient moduli versus temperature are presented in figures 5 and 6, respectively. The resilient modulus test produced a shift of -9.4 °C (-17 °F) at the 20,700 MPa

(3,000 ksi) reference modulus for the PlusRide mixture compared to the control. The tensile strength test produced a shift of -7.8 °C (-14 °F) at 2.07 MPa (300 psi). The PlusRide mixture produced lower moduli and tensile strengths at all temperatures; it thus was more resistant to low temperature cracking.

Conclusions

Both Verglimit and PlusRide rubber were found to affect various asphalt mixture properties, as summarized below.

Verglimit

- Verglimit provided Marshall stabilities and flows similar to the control.
- Verglimit mixtures had low retained tensile strength and resilient modulus ratios. Thus, Verglimit increased the moisture susceptibilities of the mixtures.
- Verglimit reduced the temperature susceptibilities of the mixtures as measured by the creep moduli, repeated load moduli, and permanent deformations and strains. The effects were slightly below 25 °C (77 °F).
- Verglimit had no effect on the resistances of the mixtures to low temperature cracking.
- Specimens containing Verglimit stored at room temperature swelled within 28 days. How this relates to field performance is unknown, although it seems to explain why there have been reports of raveling in pavements. Verglimit absorbs moisture from the air.
- Some changes to the testing procedures were required. Verglimit is water soluble so the volumetric flask method of AASHTO T209 and American Society for Testing and Materials D2041 or a volumeter must be used to determine the maximum specific gravity of the mixture. For determining bulk specific gravity, only a 1-minute period of immersion in water can be used. To mix the materials, the unheated Verglimit particles were added after the asphalt cement and aggregate were mixed; an additional 15 to 30 seconds of mixing was needed to ensure coating and a visually homogenous distribution.

PlusRide

- PlusRide mixtures had low retained tensile strength and resilient modulus ratios. The rubber particles increased the moisture susceptibilities of the mixtures.

Table 6. Resistance to rutting—creep test results: Plus Ride

	Control			PlusRide		
Creep Time	Temperature			Temperature		
(sec)	65 °F	77 °F	104 °F	65 °F	77 °F	104 °F
	Creep Modulus (ksi)			Creep Modulus (ksi)		
0.10	667	297	67	286	100	26
0.30	395	168	51	148	52	14
1.0	203	97	46	73	28	9
3.0	116	66	48	36	20	7
10.0	82	51	45	19	20	7
30.0	53	46	—	14	—	—
100.0	44	42	—	—	—	—
1,000.0	34	37	—	—	—	—
	Permanent Strain microinches)			Permanent Strain (microinches)		
0.10	19	124	535	65	167	386
0.30	56	223	733	109	540	1,561
1.0	145	429	1,002	414	1,141	3,362
3.0	283	626	1,042	968	1,879	6,592
10.0	506	870	999	2,168	2,601	12,503
30.0	628	981	—	3,040	—	—
100.0	920	1,163	—	—	—	—
1,000.0	1,619	1,765	—	—	—	—

(ksi)(6895) = (KPa) (in)(2.54)=(cm) (°F - 32)/1.8 = °C

Table 7. Resistance to rutting—repeated load test results: Plus Ride

	Control			PlusRide		
Temperature	65 °F	77 °F	104 °F	65 °F	77 °F	104 °F
Number of Cycles	Permanent Strain (microinches)			PermanentStrain (microinches)		
1	18	68	291	66	177	1,644
3	36	138	496	134	376	3,885
10	73	253	885	322	835	8,343
30	115	379	1,630	630	1,562	10,670
100	154	543	3,244	1,128	2,620	14,453
200	177	667	4,628	1,487	3,524	17,765
300	185	754	5,455	1,617	4,084	22,239
400	194	835	6,204	1,692	4,426	24,769
500	198	910	6,722	1,760	4,908	26,627
600	201	976	6,998	1,826	5,112	28,857
1,000	209	1,197	——	2,079	6,219	——
3,000	235	2,080	——	2,681	8,000	——
7,000	285	3,804	——	2,994	10,511	——
10,000	300	——	——	3,143	——	——
20,000	373	——	——	4,055	——	——
30,000	415	——	——	4,451	——	——
40,000	657	——	——	4,881	——	——
50,000	692	——	——	5,552	——	——

(in)(2.54)=(cm) (°F - 32)/1.8 = °C

- PlusRide increased the resistance to low temperature cracking and decreased the resistance to rutting. PlusRide also reduced the Marshall stabilities and creep and repeated load moduli, while it increased the flow and permanent deformations and strains.

- PlusRide specimens stored in air at room temperature developed hairline cracks by 90 days. How this relates to field performance is unknown. The rubber particles on the outer edges of the specimens also began to stick out. This swelling of the rubber particles was attributed to the absorption of asphalt hydrocarbons.

Recommendations

- Verglimit and PlusRide mixtures should be tested for moisture susceptibility and an antistripping agent used if necessary. This practice will not, however, control the inherent swelling that these deicers can cause.

- Most pavement sections in which these deicers have been incorporated are less than 2.54 cm (1 in) thick so as to reduce costs and because the additives only act at the surface of the pavement. Consequently, and because both additives had some detrimental effects on the test data, they should only be used in surface layers less than 2.54 cm (1 in) thick.

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(9) R.E. Allison. *PlusRide Asphalt Concrete Pavement,* WA-RD 130.2, Washington State Department of Transportation, Olympia, WA, January 1990.

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RECENT PUBLICATIONS

The following are brief descriptions of selected publications recently published by the Federal Highway Administration, Office of Research and Development (R&D). The Office of Engineering and Highway Operations R&D includes the Structures Division, Pavements Division, Materials Division, and Long Term Pavement Performance Division. The Office of Safety and Traffic Operations R&D includes the Intelligent Vehicle-Highway Systems Research Division, Design Concepts Research Division, and Information and Behavioral Systems Division. All publications are available from the National Technical Information Service (NTIS). In some cases, limited copies of publications are available from the R&D Report Center.

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6300 Georgetown Pike
McLean, Virginia 22101-2296
Telephone: (703) 285-2144

The Stability of Riprap Used to Protect Bridge Piers, Publication No. FHWA-RD-91-063

by Structures Division

Bridge piers subjected to floodwaters induce strong secondary currents that cause local scour holes in alluvial streambeds. Local scour holes may undermine pier foundations. A common method of protecting the streambed is installing a riprap apron. However, the effect of the secondary currents on the bed material must be known before sizing the rock protection.

Small-scale model experiments were performed in laboratory flumes to study the stability of riprap protection. Rectangular and cylindrical flow obstructions were used to model bridge piers. The influence of obstruction shape and relative depth of gravel placement, gravel size, flow depth, and approach flow roughness on near-bed velocities and critical displacement conditions was investigated.

Obstruction shape, relative depth of gravel placement, and relative flow depth were found to significantly influence near-bed velocity. Significant factors in the analysis of the critical flow conditions were obstruction shape, relative depth of gravel placement, and relative gravel size.

Limited copies of this publication are available from the R&D Report Center. Copies may also be purchased from the NTIS. (PB No. 91-236315/AS, price code: A07.)

Reinforced Soil Structures Volume I: Design and Construction Guidelines, Publication No. FHWA-RD-89-043

by Materials Division

This report presents comprehensive guidelines for evaluating and using soil reinforcement techniques to construct retaining walls, embankment slopes, and natural or cut slopes. A variety of available systems for reinforced soil including insitu soil nailing are described. Detailed guidelines are given for design of reinforced soil structures with inextensible and extensible reinforcements and soil nailing. These guidelines were developed from an extensive technical review of laboratory model, small-and large-scale centrifuge, and full-scale field tests as well as finite element numerical studies.

Limited copies of this publication are available from the R&D Report Center. Copies may also be purchased from the NTIS. (PB No. 91-197269/AS, price code: A14.)

Patching of Open-Graded Friction Courses, Publication No. FHWA-RD-89-100

by Pavements Division

This report presents the state of the practice of patching open-graded asphalt friction courses. The report is based on data collected from selected States. Current policies, practices, procedures, and methods are discussed. The performance of open-graded asphalt friction courses, patching problems associated with them, and alternatives are included. The study findings are presented together with recommendations.

Limited copies of this publication are available from the R&D Report Center. Copies may also be purchased from the NTIS. (PB No. 91-178459/AS, price code: A05.)

Fatigue Cracking of Steel Bridge Structures Volume I: A Survey of Localized Cracking in Steel Bridges 1981 to 1988, Publication No. FHWA-RD-89-166

The localized failures of the structures documented in this publication are representative of the various categories of localized failures that have occurred between 1981 and 1988. A total of 43 categories of design detail (formerly 28 categories) which contained cracking were reviewed. The representative sample provides for each bridge site a description of the structure, a summary of the cracking (which includes photographs), the known characteristics of the material and crack surface, the field measurements, and the retrofit procedures which were used to restore the cracked section and to prevent its reoccurrence elsewhere in the structure.

This volume is the first in a series. The other volumes in the series are:
RD-89-167 Volume II Fatigue Cracking of Steel Bridge Structures—A Commentary and Guide for Design, Evaluation, and Investigation of Cracking
RD-89-168 Volume III Fatigue Cracking of Steel Bridge Structures—Executive Summary

Limited copies of this publication are available from the R&D Report Center. Copies may also be purchased from the NTIS. (PB No. 91-210823/AS, price code: A16.)

Fatigue Cracking of Steel Bridge Structures, Volume II: A Commentary and Guide for Design, Evaluation, and Investigation of Cracking, Publication No. FHWA-RD-89-167

by Structures Division

The phenomena of fatigue failure at various common structural details in steel bridges is presented with suggested improvements for future design and fabrication. Fatigue damage has resulted from a large number of cyclic stresses at structural details which have low fatigue resistance. Procedures are presented for the estimate of live load stresses at bridge details and for the estimate of fatigue damages. Guidelines for examination of detected cracks are given, and procedures for the evaluation of crack propagation and fracture is suggested. The report provides background information on the historical development of experimental data, summarizes the theoretical and experimental treatment of structural details, and reviews the primary factors contributing to fatigue damage.

This volume is the second in a series. The other volumes in the series are:
RD-89-166 Volume I: Fatigue Cracking of Steel Bridge Structures—A Survey of Localized Cracking in Steel Bridges 1981 to 1988
RD-89-168 Volume III: Fatigue Cracking of Steel Bridge Structures—Executive Summary

Limited copies of this publication are available from the R&D Report Center. Copies may also be purchased from the NTIS. (PB No. 91-210831/AS, price code: A16.)

Fatigue Cracking of Steel Bridge Structures Volume III: Executive Summary, Publication No. FHWA-RD-89-168

by Structures Division

Fatigue cracking in steel bridges has become more frequent in the past decade. Among the early occurrences of cracking in the 1960's, was distortion related cracking in the stringers of suspension bridges. The 1970's resulted in fatigue cracking at a large number of details. Among them were low fatigue resistant welded details (categories E and E') such as cover-plated beams and equivalent lateral gusset plates, defective groove welds in secondary attachments such as longitudinal stiffener, lack-of-fusion in cover plate and flange groove welds and at flange penetrations of intersecting web members. These low fatigue resistant details resulted from an inadequate experimental base and an overly optimistic specification provision developed from experimental data from the 1960's. Subsequent laboratory data has verified the low fatigue strength in the high cycle region. The assumption of a fatigue limit at $2x10^b$ or 2 million cycles proved to be incorrect. Cracking due to distortion has continued to increase, affecting nearly every type of bridge. It results from small web gaps which were more frequently used with welded structures. These factors are reviewed, evaluated and assessed in the report and its companion volumes.

This volume is the third in a series. The other volumes in the series are:
RD-89-166 Volume I Fatigue Cracking of Steel Bridge Structures—A Survey of Localized Cracking in Steel Bridges 1981 to 1988
RD-89-167 Volume II Fatigue Cracking of

Steel Bridge Structures—A Commentary and Guide for Design, Evaluation, and Investigation of Cracking.

Limited copies of this publication are available from the R&D Report Center. Copies may also be purchased from the NTIS. (PB No. 91-210849/AS, Price code: A05.)

Specification for the Collection and Storage of Crash Test Data, Volume I: Final Report, Publication No. FHWA-RD-91-038; Specification for the Collection and Storage of Crash Test Data, Volume II: Users Manual, Publication No. FHWA-RD-91-039

by Design Concepts Research Division

The report presents a set of recommended specifications for the collection and storage of crash test data. The data specifications presented in this report provide guidelines for formatting the processed data, assembling data from various crash tests conducted by different testing agencies, and transmitting this data to the Federal Highway Administration for archiving.

Current procedures for collecting and storing crash test data by six leading testing agencies in the Nation were reviewed. Testing personnel from these agencies were then consulted for their views on additional data requirements, and collection and storage methodologies for three distinct test classifications: acceptance, developmental, and research. Finally, a state-of-the-art study was conducted on various data acquisition systems and storage media.

The results of the investigation were compiled in the form of a set of recommended specifications and a standard data disk. The latter contains a test information header file consisting of 65 data fields in five categories: header, appurtenance, vehicle, test conditions, and test evaluation. To facilitate the creation of this information header file, a data base application program named HEADER was developed in the dBase IV environment. Further, a second application program named Test Data Header Generator (TDHG) was developed in Quick BASIC for appending the header segment of the information file to the appropriate data channel files.

Limited copies of these publications are available from the R&D Report Center. Copies may also be purchased from the NTIS. (for RD-91-038 PB No. 91-201285/AS, price code: A03; for RD-91-039 PB No. 91-210277/AS, Price code: A03.)

Side Impact Tests of a Full-Size and Mini-Size Vehicle into a Breakaway Luminaire Support, Publication No. FHWA-RD-91-061

by Design Concepts Research Division

This report contains the results of side crash tests performed using a 1988 Ford Taurus and a 1988 Honda Civic. The cars were crashed into a three bolt slip base type breakaway luminaire support. These tests were performed at the Federal Outdoor Impact Laboratory located in McLean, Virginia. The automobiles impacted the luminaire supports at 13.41 m/s (30 mi/h) so that the vehicle aligned with the side impact dummy's pelvis with the seat in a midtrack position. In both cases the luminaire support broke away. Using the procedures specified in NCHRP 230, the occupant changes in velocity were computed. The values of the changes in velocities were less than 16 fps and similar to values obtained from earlier tests using 1979-1981 vehicles.

This publication may only be purchased from the NTIS. (PB No. 90-204561/AS, Price code: A03.)

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

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Stream Stability at Highway Structures, Publication No. FHWA-IP-90-014

by Office of Technology Application

This document provides guidelines for identifying steam instability problems at highway stream crossings and for selecting and designing appropriate countermeasures to mitigate potential damages to bridges and other highway components at stream crossings. The manual covers geomorphic and hydraulic factors that affect stream stability and provides a step-by-step analysis procedure for evaluating stream stability problems. Guidelines and criteria for selecting countermeasures for stream stability problems are summarized, and the design of three countermeasures (spurs, guide banks, and check dams) is presented in detail. Conceptual design considerations for many other countermeasures are summarized.

This publication may only be purchased from the NTIS. (PB No. 91-198788/AS, price code: A03.)

Evaluating Scour at Bridges, Publication No. FHWA-IP-90-017

by Office of Technology Applications

This document contains the state of knowledge and practice for dealing with scour at highway bridges. The procedures for designing new, replacement, and rehabilitated bridges to resist scour are presented. Procedures are given for evaluating the scour vulnerability of existing bridges as well as inspecting bridges for scour. The use of countermeasures to protect bridges evaluated as failure prone due to scour is also presented. The document replaces the Federal Highway Administration (FHWA) publication "Interim Procedures for Evaluating Scour at Bridges," which was issued with FHWA Technical Advisory 5140.20, "Scour at Bridges," in September 1988.

This publication may only be purchased from the NTIS. (PB No. 91-198739/AS, price code: A03.)

Value Engineering Study of the Repair of Transverse Cracking in Asphalt Concrete Pavements, Publication No. FHWA-TS 89-010

by Office of Technology Applications

This report summarizes the results of a cooperative value engineering study on the repair of transverse cracks in asphalt concrete pavements. The objective of the study was to optimize the expenditure of maintenance funding through an indepth study of the present methods, materials, and equipments being used, and to develop better methods, materials, equipment, and work crews for optimum and safe repair of such cracks.

This report contains recommendations and guidelines on crack preparation, materials, equipment, and timing to effect cost-effective repairs to transverse cracks in asphalt pavements. All team members agree that timely, effective crack sealing extending pavement life and reduces future maintenance costs.

This publication may only be purchased from the NTIS. (PB No. 91-127274/AS, price code: A03.)

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

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

NCP Category A—Highway Safety

A.1: Traffic Control for Safety

Title: Communicating Lane Drops to Motorists (NCP No. 4A1E0312)

Objective: After reviewing the techniques used in Texas for closing lanes, develop and test alternate signs and markings. Use motorist interviews and field tests. Determine if changes in pavement marking and signs at lane drops are an effective treatment.

Performing Organization: Texas Transportation Institute, College Station, TX 77843

Expected Completion Date: August 1993

Estimated Cost: $150,000 (HP&R)

NCP Category B—Traffic Operations

B.1: Traffic Management Systems

Title: Develop Specifications for Vehicle Classification (NCP No. 4B1A4052)

Objective: Results of the study of commercially available equipment will be used to develop specifications for automatic vehicle classification and automatic violator vehicle identification systems.

Performing Organization: California Department of Transportation, Sacramento, CA 95807

Expected Completion Date: September 1994

Estimated Cost: $335,000 (HP&R)

Title: Design, Installation, and Operation of Freeway Lane Control (NCP No. 4B1E2031)

Objective: Develop information to assist with the design, installation, and operation of freeway lane control systems.

Performing Organization: Texas A&M University (Texas Transportation Institute), College Station, TX 77843

Expected Completion Date: May 1993

Estimated Cost: $190,000 (HP&R)

B.2: Traffic Analysis and Operational Design Aids

Title: Determination of Motorist Delay in Project Specific Traffic Control Plans (NCP No. 4B2B1272)

Objective: Develop procedures to determine optimal work hours, sequencing of work operations, contractor penalties, alternative routes, and other mitigation measures during highway construction. This is a continuation study involving the University of Florida and the Texas Transportation Institute.

Performing Organization: University of Florida, Gainesville, FL 32611

Expected Completion Date: August 1993

Estimated Cost: $74,500 (HP&R)

Title: Interactive Graphics Intersection Design System (NCP No. 4B2B2082)

Objective: Extend the Interactive Graphics Intersection Design System. Add additional features to assist in analysis and design of at grade intersections.

Performing Organization: University of Texas, Austin, TX 78712

Expected Completion Date: August 1993

Estimated Cost: $186,000 (HP&R)

NCP Category C—Pavements

C.2: Evaluation of Flexible Pavements

Title: Long Term Performance Evaluation of Polymer Modified Asphalt Concrete Pavements (NCP No. 4C2A2622)

Objective: Establish a long term pavement performance monitoring program of polymer-modified asphalt concrete test sections. Test sections will be placed in different environments, with various traffic levels. Perform an evaluation to determine the critical engineering steps to predict long term pavement performance. Guidelines and specifications will be prepared to select cost effective asphalt concrete mixtures.

Six test sections were built in six districts. Cores will be obtained from these sections yearly, from 1988 to 1995. Laboratory tests include indirect tensile, resilient modulus, stability, air voids, and density. Tests will be run on recovered binder when sections show distress.

Performing Organization: University of Texas, Austin, TX 78712

Expected Completion Date: August 1992

Estimated Cost: $75,313 (HP&R)

Title: Long Term Evaluation of Stripping and Moisture Damage in Asphalt Pavements Treated with Lime and Antistripping Agents (NCP No. 4C2A2632)

Objective: Continue the evaluation of antistripping agents, including hydrated lime. Test section samples will be tested. It is expected that results will allow treatment to reduce moisture damage at a minimum cost. Three test methods, Lottman, Texas Pedestal, and Texas Boiling tests have been developed to evaluate the moisture susceptibility of AC mixtures.

Performing Organization: University of Texas, Austin, TX 78712

Expected Completion Date: August 1992

Estimated Cost: $76,383 (HP&R)

Title: Investigation of the Use of Resilient Modulus for Louisiana Soils in Design of Pavements (NCP No. 4C2B1362)

Objective: Determine the resilient moduli for Louisiana soils. Two general types of soils will be investigated under conditions representing a simulation of the physical conditions and stress states of material beneath flexible pavements subjected to moving wheel loads.

Performing Organization: Louisiana Transportation Research Center, Baton Rouge, LA 70804

Expected Completion Date: July 1993

Estimated Cost: $169,000 (HP&R)

Title: Estimation of Layer Coefficients for design of flexible Pavement Structure in Rhode Island (NCP No. 4C2B1382)

Objective: Determine initial values for layer coefficients of Rhode Island pavement materials, in order to use the 1986 AASHTO guide for the design of pavement structures. Conduct laboratory tests to determine the moduli of flexible pavement materials. The AASHTO Guide relates moduli to coefficients. An approach will be developed to account explicitly for traffic conditions, load duration, and temperature on moduli. Develop layer coefficients to represent the actual field behavior, i.e., estimate layer coefficients that when input to the AASHTO design equations will yield accurate predictions of service lives. A mechanistic model will be used.

Performing Organization: University of Rhode Island, Kingston, RI 02881

Expected Completion Date: March 1993

Estimated Cost: $180,800 (HP&R)

Title: Identify Structural Benefits of Stabilization and Updating FPS to Accommodate Stabilized Layers (NCP No. 4C2B1392)

Objective: Study the engineering properties of lime, portland cement, and fly-ash stabilized bases and subgrades and develop realistic fatigue and failure models for these layers. Models will address the change in the properties of the stabilized layer with time and traffic. Models will be added to the existing Texas flexible pavement design system.

Performing Organization: Texas Transportation Institute, College Station, TX 77843

Expected Completion Date: September 1993

Estimated Cost: $150,000 (HP&R)

C.3: Field and Laboratory Test Methods

Title: Development of an Interactive Interface for the Dynaflect Deflection Measurement System (NCP No. 4C3A1842)

Objective: Design, and install an interactive interface for the dynaflect deflection measuring system.

Performing Organization: Ohio Department of Transportation,
Columbus, OH 43215

Expected Completion Date: June 1994

Estimated Cost: $59,300 (HP&R)

Title: Improved Rut Measurement Methods for Siometers (NCP No. 4C3A1852)

Objective: Investigate the feasibility of using low cost lasers, better ultrasonic methods, or focused LED light sources with inexpensive photo detectors to obtain transverse road profiles. Determine the optimal number and spacing of these sensors, and develop the methodology for estimating rut depth from the profile. Evaluate ASTM standards for use in Texas rut depths surveys.

Performing Organization: University of Texas, Austin, TX 78712

Expected Completion Date: August 1993

Estimated Cost: $168,156 (HP&R)

Title: Evaluation of Equipment for the resilient Modulus Test (NCP No. 4C3B2042)

Objective: Assist the Alabama Department of Transportation to develop equipment and techniques for performing resilient modulus tests.

Performing Organization: University of Alabama, University, AL 35486

Expected Completion Date: September 1993

Estimated Cost: $68,500 (HP&R)

C.4: Pavement Management Strategies

Title: Development of Pavement Prediction Models for Georgia Pavements (NCP No. 4C4B3192)

Objective: Develop pavement performance models for typical Georgia pavement sections. Analyze factors that affect pavement performance to determine significance. From this information the simplest and most efficient models will be developed and verified. Review the work performed by other States. The general forms of models will be calibrated with Georgia data. Next, the most significant factors that affect performance will be selected and a stratified (5 percent) sample of pavements will be chosen to analyze. Finally, collect detailed data and use the SAS statistical package to calibrate existing models or develop new models.

Performing Organization: Georgia Department of Transportation, Atlanta, GA 30334

Expected Completion Date: January 1993

Estimated Cost: $70,000 (HP&R)

Title: Truck Pavement Interaction (NCP No. 4C4A1192)

Objective: Study how different truck suspensions, single versus dual tires, loads and inflation pressure affect pavement response. Study how pavement roughness causes truck damage. Products will include an instrumented pavement test facility at PACCAR to measure pavement responses.

Performing Organization: University of Washington, Olympia, WA 98501

Expected Completion Date: December 1991

Estimated Cost: $73,800 (HP&R)

NCP Category E—Materials and Operations

E.1: Asphalt and Asphaltic Mixture

Title: Evaluation of NCHRP Asphalt Mixture Analysis (AAMAS) (NCP No. 2E1B2252)

Objective: Evaluate the NCHRP asphalt aggregate mixture analysis system (AAMAS). The procedures will be verified using mixtures with know field performance.

Performing Organization: Federal Highway Administration, McLean, VA 22101-2296

Expected Completion Date: September 1992

Estimated Cost: $55,000 (staff study)

Title: Continued Monitoring of Pavement Test Section (NCP No. 4E1D2022)

Objective: Data has been collected and stored for some 40 pavement research sections throughout the State over the past 2 years. To better understand pavement performance, data will continue to be collected until the sections are modified (seal coated, overlaid, reconstructed, or taken out of service) incorporated into the existing data base. Relationships will be developed between performance and the measured physical properties.

Performing Organization: Center for Highway Research, Austin, TX 78701

Expected Completion Date: September 1995

Estimated Cost: $54,000 (HP&R)

Title: Deformation of Surface and Binder Courses (NCP No. 4E1D2122)

Objective: Determine the relative resistance of Arkansas surface and binder asphalt mixes to rut development. Develop recommendations relative to the practice of permitting surface mixes to be substituted for binder mixes.

Performing Organization: University of Arkansas, Little Rock, AK 72204

Expected Completion Date: March 1993

Estimated Cost: $90,776 (HP&R)

E.2: Cement and Concrete

Title: Special Concrete Design and Construction Methods for Intersections (NCP No. 4E2C1302)

Objective: Develop guidelines for the design and construction of concrete intersections to meet traffic opening schedule requirements and to minimize closures. Also develop an improved understanding and predictive models of pavement performance as they are affected by the mix design and construction methods.

Performing Organization: Texas Transportation Institute, College Station, Texas 77843

Expected Completion Date: August 1993

Estimated Cost: $198,000 (HP&R)

E.3: Geotechnology

Title: Rockfall Mitigation Measures (NCP No. 4E3B0822)

Objective: Identify and classify the common types of rockslope instability in Kentucky. Identify the causes of rockfall, examine the historical record of rockfalls in Kentucky and establish the framework for implementing a statewide rockfall hazard rating system.

Performing Organization: University of Kentucky, Lexington, KY 40506

Expected Completion Date: October 1996

Estimated Cost: $300,000 (HP&R)

E.4: Paints and Coatings for Highways

Title: Removing Lead-Based Paint from Bridge (NCP No. 4E4A0212)

Objective: Formulate guidelines that stipulate levels of waste containment for different bridge environments. Development of processing documentation and procedures to assure adherence to those guidelines would also be derived. Develop specifications and procedures for containment and disposal of removed lead-based paints.

Performing Organization: University of Kentucky, Lexington, KY 40506

Expected Completion Date: December 1992

Estimated Cost: $118,000 (HP&R)

E.5: Highway Maintenance

Title: Continued Monitoring of Seal Coat Test Sections (NCP No. 4E5C3462)

Objective: Monitor 59 pavement sections that were surfaced with various seal coats. Tests include skid testing, texture measurements, and conditions surveys at least once each year and when possible just prior to modification (i.e., resealing, overlay, out of service, reconstruction). Increased knowledge of seal coat field performance should establish better relationships among performance and measurable physical properties and give the engineer more confidence in making decisions related to materials selection.

Performing Organization: University of Texas, Austin, TX 78712

Expected Completion Date: September 1995

Estimated Cost: $67,000 (HP&R)

Title: Use of Micro-Surfacing in Highway Pavements (NCP No. 4E5C3472)

Objective: Develop draft specifications, mix design requirements, and quality assurance requirements for polymer modified emulsified asphalt slurry seals (micro-surfacings). The developed procedures will be tested in the field and adjusted as needed. Training programs which address the procedures will be developed, to include video training if necessary.

Performing Organization: Texas Transportation Institute, College Station, TX 77843

Expected Completion Date: August 1994

Estimated Cost: $200,000 (HP&R)

E.7: Environmental Design

Title: Air Pollution Implications of Urban Transportation Investment Decisions (NCP No. 4E7A2312)

Objective: Support the efforts of the Texas Department of Highways and Public Transportation to comply with the current Federal and Texas Clean Air Acts. Means of attaining the goals of this effort are summarized by reviewing the potential impacts of the Clean Air Act of 1990, transportation control measures, and synthesizing other State policies and practices on mobile source emissions. The effort will also assess the contributions of mobile source emissions on urban air quality, the roles and vehicles and fuels in reducing mobile source emissions, and the available analysis tools for estimating mobile source emissions and VMT. The effort will give improved analysis for estimated emissions and VMT.

Performing Organization: Texas Transportation Institute, College Station, TX 77843

Expected Completion Date: August 1994

Estimated Cost: $330,000 (HP&R)

Title: Normalizing Traffic Noise Measurements for the Effects of Meteorology (NCP No. 4E7B1934)

Objective: Develop a simple site-specific field procedure to adjust noise levels measured at any wind direction and speed up to the presently used criterion. Develop an algorithm that allows making the adjustment procedure, thus saving time and cost. Measurements are to be made at highways of various widths and configurations such fill and cut sections.

Performing Organization: California Department of Transportation, Sacramento, CA 95807

Expected Completion Date: September 1994

Estimated Cost: $116,442 (HP&R)

Title: Wetlands - Policies and Procedures (NCP No. 4E7C4416)

Objective: The State highway agency must satisfy several oversight agencies during planning, designing, and construction of a project concerning wetlands. Policy options include avoiding, minimizing, rectifying, reducing impacts to wetlands. Innovative procedures for both on and off site have to be considered. Policy alternatives will be evaluated. Determine the most suitable wetland process/procedures to be used in planning, designing, and constructing highway project in compliance with State and Federal guidelines for wetlands. Each wetland may have to be addressed case by case.

Performing Organization: Texas Transportation Institute, College Station, TX 77843

Expected Completion Date: August 1992

Estimated Cost: $87,000 (HP&R)

E.8: Construction Control and Management

Title: New Evaluations of Liquidated Damages, Motorist Liquidated Damages, and Percent Retainage (NCP No. 4E8C2202)

Objective: Examine alternative procedures for optimizing the completion of highway projects. Procedures to be considered will include current practices (conventional and incentive/disincentive), alternative percent retainage provisions, and higher liquidated damages based partially on user costs and/or impacts on local businesses.

Performing Organization: Texas Transportation Institute, College Station, TX 77843

Expected Completion Date: August 1993

Estimated Cost: $194,000 (HP&R)

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Section 5164 of the Omnibus Trade and Competitiveness Act of 1988 (PL 100-418) provides that each agency of the Federal Government, before the end of FY 1992 implement the use of the metric system of measurements in its procurements grants and other business related activities. The FHWA has developed the following schedule for conversion.

Program elements/activities	Target date
I. Develop FHWA metric conversion plan	May 1991
II. Initiate revision of pertinent laws and regulations that serve as barriers to metric conversion	May 1992
III. Conversion of FHWA manuals, documents, and publications	May 1994
IV. Data collection and reporting	May 1995
V. Direct Federal and Federal-aid construction contracts	September 30, 1996

As a first step in the conversion process, commencing October 1, 1991, the Offices of the Associate Administrator for Research and Development will require all future reports be prepared using metric units.

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