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Safer Guardrail/Bridge Rail Transition Designs Have Been Developed

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COVER:
Guardrail/bridge rail transitions are a critical link in roadside hardware. Crash tests of the transitions have been performed to identify the problems with different transitions and to develop a design that will better protect motorists.

IN THIS ISSUE

Articles

Departments

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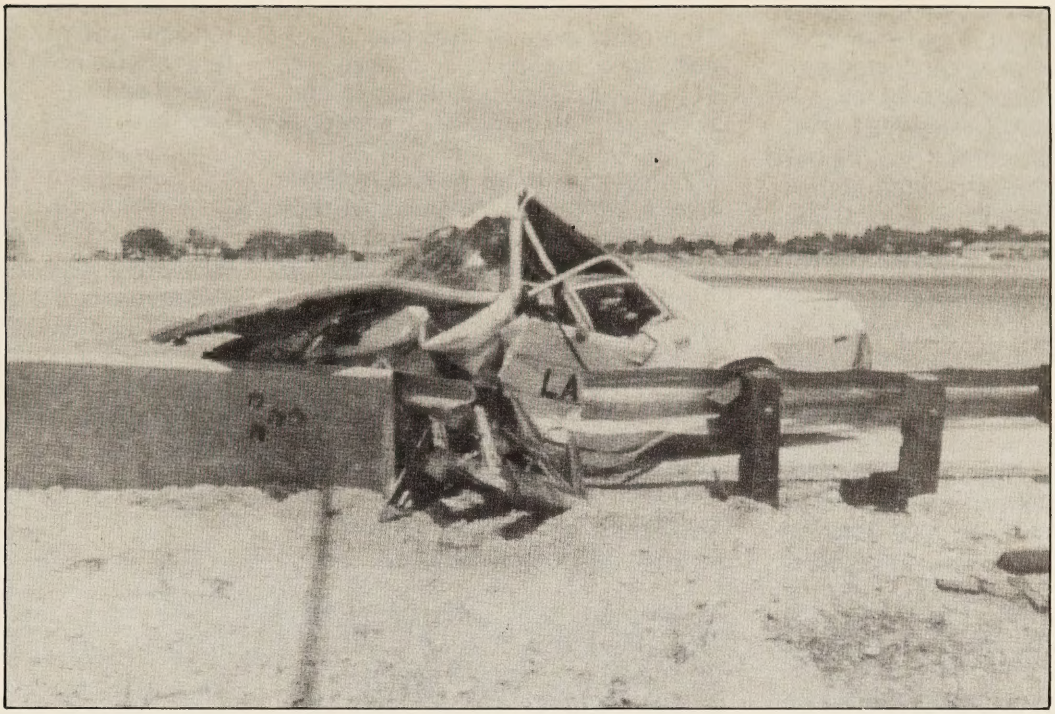


Crash Testing Bridge Rails and Transitions

by
Charles F. McDevitt

Introduction

A crash test, or full-scale test, is a controlled experiment for assessing the performance and behavior of a roadside safety appurtenance such as a bridge rail. A crash test can also be thought of as a staged accident that provides insight into what will happen in real-world accidents. Since it is not economically feasible to investigate how a safety appurtenance will behave in all possible accidents, the crash test vehicles and conditions represent practical worst cases.

This article briefly describes crash testing of bridge rails and transitions, and the lessons learned from recent crash tests.

Crash Testing of Bridge Rails—History and Background

Since the mid-1960's, the principal vehicle used for testing the strength of guardrails, median barriers, and bridge rails has been a full-size sedan with a curb weight of 4,500 lb (2.04 Mg). The test conditions of 60 mi/h (97 km/h) and a 25 degree impact angle are considered severe compared to nearly all passenger car accidents. This classic strength test is considered by many to be a surrogate test for heavy vehicles as well, since it provides some barrier reserve strength for heavy vehicle accidents.

In 1974, the first recommended standard crash test and evaluation procedures for all roadside safety structures were published as National Cooperative Highway Research Programs (NCHRP) Report 153. (*1*)[1] The 1977 American Association of State Highway and Transportation Officials (AASHTO) Barrier Guide endorsed these procedures and listed as operational barrier systems only those barriers (including bridge rails) judged as essentially meeting these evaluation criteria. (*2*) (Operational systems were described as those having both crash test and field performance experience.) Of the five bridge rails listed in the barrier guide, however, only one—the New Jersey safety shape BR1—had been evaluated for the 2,250-lb (1.02 Mg) vehicle 60 mi/h (97 km/h) l5-degree angle test shown in the NCHRP report. (The 2,250-lb (1.02 Mg) Chevrolet Vegas, Ford Pintos, etc., were the small cars of the 1970's.)

[1]Italic numbers in parentheses identify references on page 70.

Beginning in 1978, a series of 21 full-scale tests were conducted on five different bridge rails by the Texas Transportation Institute (TTI). (*3*) These tests were to see how well bridge rails designed as per AASHTO bridge specifications performed when hit by small cars and 4,500-lb (2.04 Mg) full-size sedans. Other tests were conducted with 20,000-lb (9.07 Mg) and 32,000-lb (14.51 Mg) buses to investigate the bridge rails' upper performance limits. While the three bridge rails tested with buses contained the buses, the vehicles rolled over on their sides on the roadway because the rails were too low.

The bridge railings selected for testing had relatively open profiles in order to assess AASHTO railing geometrics requirements. Based on the tests with 2,250-lb (1.02 Mg) Vega sedans and 1,800-lb (0.82 Mg) Honda Civic sedans, it was evident that the requirements were not adequate to prevent bumpers and wheels from snagging on bridge rail posts. To address this problem, the principal investigator, Dr. Gene Buth, developed an empirically based relationship between the rail depth and the clear distance from the rail to the bridge deck. (*3*) Further, since the evaluation criteria for judging the results of crash tests in NCHRP Report 153 and its successor, Transportation Research Circular 191, were not adequate for bridge rails, he proposed new performance standards. (*4*) These standards were based upon the maximum lateral and longitudinal vehicle accelerations measured during a full-scale test.

In an effort to develop comprehensive performance standards for bridge rails and crashworthy designs that could be recommended for State use, the Federal Highway Administration (FHWA) contracted with the Southwest Research Institute (SwRI) to conduct another series of bridge rail tests. These tests, begun in 1983, were performed on the best of six different types of bridge rail designs selected from a list of 166 approved State standard plans. (*5*)

To provide more realistic test conditions, the principal investigator, Mr. Maurice Bronstad, mounted the bridge rails on simulated bridge deck edges which were reinforced in accordance with the State's typical reinforcing details. Since bridge deck cracking was observed in two of the tests, it was concluded that, to avoid masking-out failure modes and deficiencies in deck reinforcing, bridge rail crash tests should be carried out on simulated bridge deck edges. This is particularly true for tests with heavy vehicles. Thus, these and other tests have shown that the current guidance for designing deck edge reinforcing is not completely satisfactory.

Another problem was hood snagging, observed in a test with a 4,500-lb (2.04 Mg) car hitting a three-rail bridge rail at 60 mi/h (97 km/h) and 25 degrees. Hood snagging had, in fact, previously been observed in development tests of an approach rail and a retrofit rail for old, narrow through truss bridges. (*6*)

Results of the recently completed SwRI contract were presented at the last annual meeting of the Transportation Research Board. (*7*) A final report will be issued later this year. Based on the study's observations, Mr. Jarvis Michie last year proposed new performance standards/criteria for evaluating crash test data, as well as a multiple performance-level test matrix for bridge rails. Further, Mr. Maurice Bronstad has proposed new railing geometrics criteria to minimize snagging problems. (Figure 1.)

A major outcome of these test programs was a 1986 memorandum to FHWA field offices. The memorandum contained drawings of 22 bridge rails that had been successfully crash tested, and stated that, ". . . the information already gained from recent crash test programs is substantial and should be considered in railing designs used for new and reconstructed bridges on Federal-aid projects. . . . Other bridge railing designs should be successfully crash tested in accordance with NCHRP 230 criteria (or equivalents) before their use on future Federal-aid projects is approved." (*8*)[2]

Highway Planning and Research Program Pooled-Fund Study

In 1986, a Highway Planning and Research (HPR) program pooled-fund study on bridge rails was initiated to meet the needs of the States. Twenty-four highway agencies now are participating in this study:

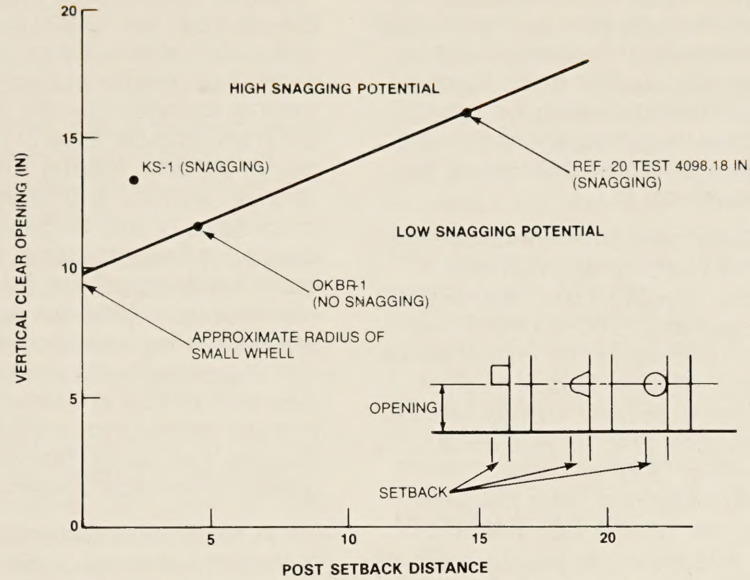
Arizona	Iowa	Ohio
California	Kentucky	Oklahoma
District of Columbia	Louisiana	Oregon
	Michigan	Pennsylvania
Florida	Minnesota	South Dakota
Hawaii	Nevada	Texas
Illinois	North Dakota	Washington
Indiana		

The 4-year study calls for Texas Transportation Institute (TTI) to conduct 38 bridge rail and 12 transition tests. So far, the following bridge rail designs have been selected and tested:

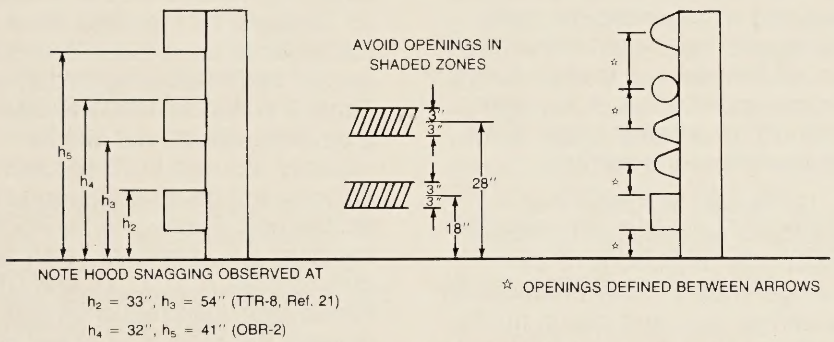
- A continuous vertical concrete wall.
- The Illinois 2399 bridge rail (2-rail, curb-mounted).
- The F-shape bridge rail.

State representatives are now voting (September1987) on additional bridge rail and transition designs to be crash tested. Texas Transportation Institute will begin crash testing of these rails and transitions in December 1987. After these crash tests are performed, the panel will vote on the next bridge rail and transition designs to be tested.

[2]NCHRP Report No. 230 replaced NCHRP Report No. 153 and Transportation Research Circular No. 191 in 1981.



(a) Lower opening considerations (based on Honda Civic front wheel for test conditions up to 60 mi/h (97 km/h) and 20 degrees).



(b) Upper opening considerations (based on 4,500-lb (2.04 Mg) car, 60 mi/h (97 km/h) impact at 25 degrees, rigid barriers).

Figure 1.—New railing geometrics criteria.

Test Vehicles

NCHRP Report No. 230 recommended that both 1,800-lb (0.82 Mg) cars—the new small cars of the 1980's—and the older 2,250-lb (1.02 Mg) cars be used in testing all longitudinal barriers. (*9*) The test conditions were to be 60 mi/h (97 km/h) at an impact angle of 15 degrees. Since there was insufficient experience with 1,800-lb (0.82 Mg) test cars, a longitudinal barrier that passed a test with a 2,250-lb (1.02 Mg) vehicle would still be considered acceptable even though it had failed a test with an 1,800-lb (0.82 Mg) vehicle.

NCHRP Report No. 230 has a supplementary test matrix containing an even more severe test: An 1,800-lb (0.82 Mg) car hitting at 60 mi/h (97 km/h) and 20 degrees. Tests conducted under NCHRP Project 22-4, "Performance of Longitudinal Traffic Barriers," and other studies have demonstrated that the operational guardrail and median barrier systems can successfully redirect an 1,800-lb (0.82 Mg) car after a 60-mi/h (97 km/h) impact at 20 degrees. Further, SwRI's recent bridge rail tests have demonstrated that good bridge rail designs also can redirect an 1,800-lb (0.82 Mg) car under these test conditions. (*5, 10*)

It has been found that tests with 1,800-lb (0.82 Mg) cars at 15-degree impact angles are not discerning predictors of occupant injury risk. (*10*) Consequently, recent test programs have used 1,800-pound Honda Civic or Volkswagen Rabbit sedans hitting at 60 mi/h (97 km/h) and 20 degrees.

The supplementary test matrix in NCHRP Report No. 230 also suggests that tests be conducted with 20,000-lb (9.07 Mg) and 40,000-lb (18.14 Mg) buses and with 80,000-lb (36.29 Mg) tractor-trailers. Some of the tests in the supplementary matrix are based upon a research study to develop multiple-service-level highway bridge railing selection procedures that were documented in NCHRP Report No. 239. (*11*)

NCHRP Report No. 230 further recommends that test vehicles be not more than 6 model-years old. Because of vehicle downsizing, full-size sedans that can be ballasted up to 4,500 lb (2.04 Mg) have not been available since 1979.

Since the heaviest full-size cars now weigh about 3,600 lb (1.63 Mg) consideration was given to using a Chevrolet Malibu or similar car as a test vehicle. (*12*) An attempt was made to use a 3,400-lb (1.54 Mg) class car ballasted up to 4,500 lb (2.04 Mg) as a test vehicle: This was unsuccessful because the front end of the overloaded vehicle collapsed upon impact. Other tests with 3,400-lb (1.54 Mg) cars on bridge rails with safety shape or open profiles, however, were very successful. No hood snagging or other problems were observed. The 3,400-lb (1.54 Mg) car therefore was not selected as a test vehicle, since it did not have any unusual characteristics or critical properties.

A replacement for the classic strength test with a 4,500-lb (2.04 Mg) car hitting at 60 mi/h (97 km/h) 25 degrees has been identified in a study by Mr. Malcom Ray at SwRI. (*12*) It has been concluded that a 3/4-ton pickup truck, when ballasted up to 5,400 lb (2.45 Mg) with steel plates and when hitting a barrier at 65 mi/h (105 km/h) and 20 degrees, produces the same barrier deflection, the same forces on the vehicle and its occupants, and at the same exit trajectory as the 4,500-lb (2.04 Mg) car test. A combination concrete safety shape bridge rail, or an F-shape bridge rail, and a 27-in (0.69 m) high, G4(ls) guardrail have successfully redirected a 5,400-lb (2.45 Mg) pickup truck under these test conditions.

Lessons Learned from Bridge Rail Crash Tests

The following information may be useful in designing safer bridge rails:

- There is no correlation between the 10,000-lb (44.5 N) static load prescribed by AASHTO bridge specifications and the peak impact force imparted to a bridge rail by a 4,500-lb (2.04 Mg) car. Tests with a 4,500-lb (2.04 Mg) car hitting a vertical instrumented wall at 60 mi/h (97 km/h) and 25 degrees show the peak impact force is about 60,000 lb (266.9 N).

- The post of the Texas T101 bridge rail and the reinforcement in the 7.5-in-thick deck have a strength of about 40,000 lb (177.9 N). (*13*) A thicker, more heavily reinforced slab edge should be used to avoid high maintenance costs from deck cracking problems.

- A barrier's "effective height" is more important than its overall height in determining whether or not a bridge rail (or other traffic barrier) can contain and redirect a vehicle. Effective height depends upon the shape and relative stiffness of the rail elements. (*14*)

- Tests have been conducted on a vertical instrumented wall with vehicles ranging from 1,800-lb (0.82 Mg) cars to 32,000-lb (14.51 Mg) buses; information on impact forces and the effective barrier heights required to redirect various vehicles is available. (*14, 15*) Additional tests on a 90-in (2.29 m) -high instrumented wall have just been conducted by TTI with a 40,000-lb (18.14 Mg) bus, an 80,000-lb (36.29 Mg) tractor-trailer and an 80,000-lb (36.29 Mg) tractor-gasoline tanker. The test data are currently being analyzed to determine the peak forces and their height above the ground. However, it is clear from the preliminary results that the peak force occurs when the rear of the vehicle strikes the barrier.

- Dr. Robert M. Olson has developed an equation to provide reasonable predictions of the average impact forces produced by passenger cars and pickup trucks. (*16*)[3] However, this equation underestimates the peak impact forces produced by automobiles and greatly underestimates the impact forces from longer or heavier vehicles.

- Even though some bridge rails did not meet the 10,000 lb (44.5 N) static load criteria, they passed crash tests. This was because of the reserve strength inherent in steel and concrete, and because of the high rate of dynamic loading. Ultimate strength analysis and design procedures should be used to develop more predictable, better balanced-designs.[4] Dr. T.J. Hirsch has developed yield-line analysis procedures for determining the ultimate strength of concrete bridge rails with vertical or safety shape profiles; he has also applied plastic analysis and design procedures to metal rail and post systems. (*17*)

- In some tests, the strengths of the posts in the impact zone were reduced because the aluminum nuts stripped off of the anchor bolts. In other tests, galvanized nuts slipped off because the threaded holes were cut larger to allow room for the anchor bolt's galvanized coating. Table 3 in ASTM A563 indicates that a tapped oversize nut has its capacity reduced to 75 percent of that of a nut on an ungalvanized anchor bolt. Further ASTM A563 specifies five grades of heavy hex nuts (grades A, B, C, D, and DH). To remedy this load reduction and develop the proof load stress of a galvanized anchor bolt, a higher grade of nut can be specified (e.g., a grade C nut can be specified instead of a grade A nut).

[3] See 1977 AASHTO Barrier Guide, page 289.

[4] See "Safer Bridge Railings," page 125.

- Conventional cut steel washers should be used at post anchorages because cast steel washers have shattered during crash tests. (*18*)

- The traffic side of bridge rails and their end blocks must be free of projecting elements or surfaces that can snag a vehicle.

- If one or more posts break away in a metal rail and post system, the bridge rail may still be able to prevent penetration and redirect the vehicle if the railings have tensile continuity.

- Full-scale tests have shown that a bridge rail, generally perceived as a rigid structure, can deflect as much as 4 ft (1.2 m) and still safely redirect the test vehicle (provided there is nothing protruding from the deck edge that can snag the front wheel). This deflecting bridge rail concept has been used for the Texas T6 bridge rail and the wood and steel post SL1 bridge rails shown in NCHRP Report 239, which are intended for "low service level" applications. (*11*)

- Crash tests conducted at 40 mi/h (64 km/h) with automobiles and school buses on a sharply curved offramp with high superelevation and downgrade have shown that concrete safety shape bridge rails can be installed either vertical or perpendicular to a superelevated roadway. (*19*)

- Concrete safety shape barriers are designed to lift cars to reduce the friction between the tires and the roadway: This helps in turning and redirecting vehicles. Since the lower concrete surface of a safety shape acts like a ramp, this lifting action increases the potential for small cars to become unstable and to rollover. The performance of the New Jersey safety shape, however, is satisfactory because it has successfully redirected an 1,800-lb (0.82 Mg) Honda Civic sedan after a 60-mi/h (97 km/h) impact at 20 degrees. Since the ramp on the F shape is lower, it will even further reduce the potential for vehicle instability problems. (*20*)

- A vertical wall will provide the maximum reduction in rollover potential since all four vehicle wheels tend to remain in contact with the ground. Recent research on barriers suggests that the ideal bridge rail should have a vertical face, or railings with faces in the same vertical plane, and that it should deflect backwards about 3 to 6 in (76.2 to 152.4 mm). (*21*) The deflection will cushion the impact and result in the vehicle being redirected nearly parallel to the barrier.

Retrofit Bridge Rails

New York State has developed a curb-mounted thrie beam on strong post retrofit, a 6-in by 6-in (152.4 by 152.4 mm) box beam retrofit, and a 10-gage thrie beam retrofit for upgrading discontinuous metal panel railings. (*22, 23, 24*) A tubular thrie beam on collapsing tube retrofit and a SERB retrofit have been successfully tested with vehicles ranging from 1,800-lb (0.82 Mg) cars to 40,000-lb (18.14 Mg) buses. (*20, 24*) Two retrofit bridge rails have been developed for narrow truss bridges. (*6*) All of these retrofit designs have crash-tested transitions to the approach guardrail, so they can be installed as complete systems.

Transitions

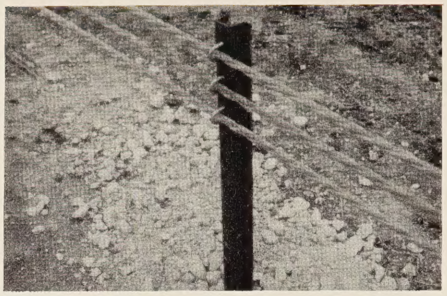
The various guardrail-to-bridge rail transition designs now in use were reviewed. (*25*) The best and most typical designs were selected for crash testing. Since current transition designs did not perform very well, retrofit designs and new transition designs were developed and successfully crash tested with 4,500-lb (2.04 Mg) cars. (*25, 26*) An FHWA Technical Advisory on transitions will be sent to field offices in a few months. The key features of these transition designs are:

- Standard posts with blockouts (4 spaces at 1.5 ft (0.46 m)) at the bridge end, followed by the same posts spaced at 3 ft, 1½ in (0.95 m). It was shown by computer simulation that using larger posts near the bridge end was not as effective as reducing the spacing of standard posts.

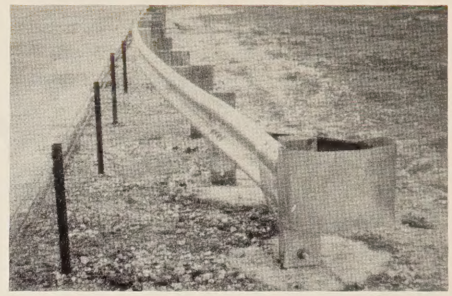
- One W-beam panel (12 ft, 6 in (3.81 m)) used as a lower rub rail on straight wingwalls or parapets.
- W-beam used with single collapsing tube blockout when attached to a tapered wingwall or parapet. Wingwalls should have a 4:1 taper.
- Thrie beam on both straight and tapered wingwalls.
- The upper W-beam rail or thrie beam rail panel at the bridge end is doubled (nested) to reduce local deformations.

South Dakota's new 27-in high (0.69 m), 3-cable guardrail has hat-section posts made of rerolled rail steel. (*25*) These 4-lb/ft posts are (5.95kg) significantly less expensive than S3 by 5.7 posts. Consequently, the guardrail costs about $1 per ft or 10 percent less than the G1 guardrail. The G1, 3-cable guardrail and this modified guardrail both deflect about 11 ft when hit by a 4,500-lb (2.04 Mg) car at 60 mi/h (97 km/h) and 25 degrees. This makes it very difficult to make a transition between a 3-cable guardrail and a W-beam guardrail, or a bridge end.

To make the transition, the post spacing was reduced, and a breakaway cable terminal (BCT) with a 4-ft (1.22 m) flare was placed behind the 3-cable guardrail. A series of tests were conducted with 4,500-lb (2.04 Mg)/ cars hitting the side and nose. It was found that the BCT did most of the work in redirecting the 4,500-lb (2.04 Mg) car when it hit in the transition zone. Post spacing alongside the BCT, therefore, was increased. Concrete foundations were used instead of steel foundations for the first two posts of the BCT to prevent rotations of these posts that might cause wheel snagging problems. The end anchorage of the 3-cable guardrail was located out of the way behind the W-beam guardrail. The State's 3-ft (0.91 m) diameter concrete foundation was used because other tests had demonstrated that it had adequate resistance to pull out. These features provided an acceptable solution to a very difficult design problem.

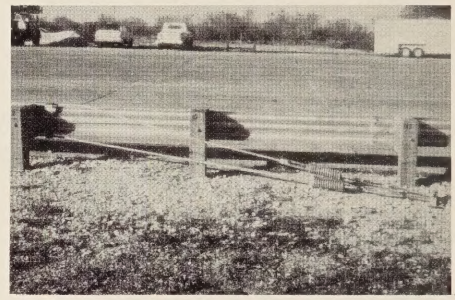


(a) Close-up of guardrail post.



(b) Transition to a W-beam guardrail.

(c) Transition to a W-beam guardrail.



(d) Cable anchorage behind guardrail.

Details of the South Dakota three-cable guardrail.

Conclusion

A collision with a barrier system is a very complex event. The dynamic loads, the large deflections of the barrier and the vehicle, and materials that are stressed into the elasticplastic region make rigorous analysis very difficult. Even seemingly insignificant details such as washers, can make the difference between successful performance and failure. At present, full-scale tests are the best way of verifying that a traffic barrier or other roadside safety device performs as it was intended by the designer.

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A Further Statistical Treatment of the Expanded Montana Asphalt Quality Study

by
Joseph A. Zenewitz and Kim Thanh Tran

Introduction

The following study's origin is attributed to a Desert Research Institute report, "Statistical Analysis of the Expanded Montana Asphalt Quality Study" by Robert R. Kennison received for review and critique. (*1*)[1] In the course of furnishing both as requested, sufficient interest was kindled in the group for further statistical consideration of the data. Our approach shall, for the most part, exploit the excellent chemical data relating to the percentages of large- and small-sized molecules obtained for each of the asphalts recovered from each of the pavement samples by Jennings et al. and listed in appendix A of Kennison's report.

In general, large-sized molecule content (generally branch-chained compounds) relates to stiffness or hardness of an asphalt as compared to the small-sized molecule content, generally paraffinic (aliphatic), which contributes a softening effect or property to asphalts. The paraffins have been found to help resist oxidative hardening as reported by Zenewitz and Welborn. (*2*) Actually, some a priori considerations concerning asphalts with a prevalent large-size or small-size molecule content can be addressed, relating to climatic and traffic conditions. For instance, paving technologists in hot climatic areas would be apt to use harder-grade asphalts containing a greater than usual level of large molecules to prevent rutting, especially if the level of traffic warrants this. On the other hand, for cold areas, paving engineers would choose softer-grade asphalts with small molecules to avoid hardening and cracking. It also is expected that the level of traffic per day would affect the choice of asphalts.

It was hopeful at the outset of affirming the foregoing a priori considerations and also hold liable any significant departures from average of the levels of large-size or small-size molecules in pavement asphalts for cracking of various types, rutting, and the rated condition of pavement with the help of a computer, STAT computer program, and a computer oriented chemist.

[1]Italic numbers in parentheses identify references on page 81.

Preliminary Considerations

Initially, the computer and personal examination, was directed toward the distribution and associated statistics of the reported percentages of the High Performance Liquid Chromatography (HPLC) detected large molecules in the asphalts obtained from pavement samplings in 15 States. This also was done for the detected small molecule percentages for these asphalts. Considering the near normal and symmetrical quality of both percentage distributions, a decision was made to compare the arithmetic means and standard deviations of either or both molecular size percentages associated with any particular problem, e.g., rutting, cracking, condition, or factor (climate, traffic), and compare each with the overall average and standard deviation for the applicable size molecule. A t-test, a part of the STAT computer program, was used to determine significant increases or decreases of either average molecular size content at a 5 percent chance occurrence confidence level relating to the problem or factor.

The t-test is a frequently-used statistical procedure for establishing significant differences between averages (arithmetic means). It is based on and uses values of a symmetrical distribution which vary with the degree of freedom (the number of items involved). Although the t-distribution is symmetrical, it differs from the normal distribution in that at the lower degrees of freedom (N-1), up to at least 30, it is less peaked than the normal distribution and is considered more representative of such data. With its greater flatness, it has more area in its tails for equivalent degrees of freedom when compared to the normal. The characteristic tends to make for a tougher test than for the normal z-Test at certain levels for significant differences. Further, as the number of degrees of freedom increases, the t-distribution approaches a normal distribution. The following formula is contained in the STAT computer program for obtaining the various t-Values discussed in this report:

$$t = \frac{\bar{X}_1 - \bar{X}_2}{\sqrt{\left(\frac{\Sigma(X_1 - \bar{X}_1)^2 + \Sigma(X_2 - \bar{X}_2)^2}{N_1 + N_2 - 2}\right)\left(\frac{1}{N_1} + \frac{1}{N_2}\right)}}$$

Age of Pavement and the Associated Large- and Small-Molecule Content of its Asphalt

Considering the aspect of asphalt change through the years that has been of concern to asphalt technologists, the data contained in table 1 was plotted to obtain figure 1. In this figure, some interesting ascendencies and descendencies of levels of both the large- and small-sized molecule content of asphalts in opposition to each other are discerned.

According to the plot, in the newer pavements going back to 7-year old pavements and from 9- through 16-year olds, the large-molecule content of asphalts was the greater. However, in pavements between 16 and 33 years old, the smaller-size molecule is in the ascendancy in the asphalts derived from these older pavements. It would appear that approximately 16 years back from the sampling date, a change occurred in asphalt composition. Of course, another explanation could be that the persistent and surviving pavements started with asphalts richer in smaller molecules, paraffins, while pavements constructed with asphalts with large molecules failed and are not available for sampling.

Table 1.—Age of pavement vs $\bar{X}$ of large and small molecules and their SDs and SEs

			Large Molecules %			Small Molecules %		
Age Yrs.	Number	$\bar{X}$ Age	$\bar{X}$	SD	SE	$\bar{X}$	SD	SE
1 + 2	39	1.590	28.438	6.998	1.121	28.238	5.139	.823
2 + 3	57	2.596	29.386	5.858	.776	27.395	5.346	.693
3 + 4	71	3.507	31.500	6.533	.775	26.017	5.385	.639
4 + 5	84	4.571	31.004	6.853	.746	26.888	4.873	.532
5 + 6	108	5.556	31.756	7.110	.684	26.282	6.887	.663
6 + 7	105	6.429	31.125	8.033	.714	26.606	7.747	.756
7 + 8	79	7.430	26.949	7.901	.889	30.072	6.810	.766
8 + 9	80	8.575	27.174	7.752	.867	30.171	6.992	.782
9 + 10	111	9.586	29.929	8.046	.764	28.387	7.199	.683
10 + 11	102	10.363	30.000	7.274	.720	28.635	6.274	.621
11 + 12	64	11.422	30.481	5.797	.725	27.816	4.694	.587
12 + 13	46	12.413	32.885	6.575	.969	26.026	4.829	.712
13 + 14	52	13.635	32.646	5.810	.806	25.596	4.324	.600
14 + 15	47	14.298	31.928	5.659	.825	25.064	4.305	.628
15 + 16	43	15.674	29.202	6.767	1.032	27.386	4.751	.724
16 + 17	41	16.293	28.368	6.494	1.014	28.368	4.897	.765
17 + 18	31	17.613	25.219	8.256	1.483	29.987	8.358	1.501
18 + 19	22	18.136	23.236	7.823	1.668	31.127	8.823	1.881
19 + 20	11	20.	24.318	3.238	.976	32.209	5.133	1.548
20 + 21	12	20.333	25.375	3.280	.947	32.742	4.302	1.242
21 + 22	7	21.429	27.271	1.276	.482	30.700	1.404	.531
22 + 23	5	22.400	24.380	4.358	1.949	32.840	4.022	1.799
23 + 24	8	23.750	26.300	7.533	2.663	31.138	5.907	2.088
24 + 25	15	24.600	28.900	7.431	1.919	27.533	5.903	1.524
25 + 26	10	25.100	28.260	7.943	2.512	27.460	6.693	2.117
26 + 27	9	26.889	25.967	4.902	1.634	32.211	2.594	.865
27 + 28	9	27.111	25.322	6.059	2.019	33.022	4.525	1.508
28 + 29	6	28.833	22.533	6.447	2.632	33.467	5.920	2.429
29 thru 45	11	32.273	23.309	3.780	1.140	32.718	2.892	.872

$\bar{X}$ = arithmetic mean (average)
SE = standard error of mean
SD = standard deviation
– a scatter about the mean measurement of the group members

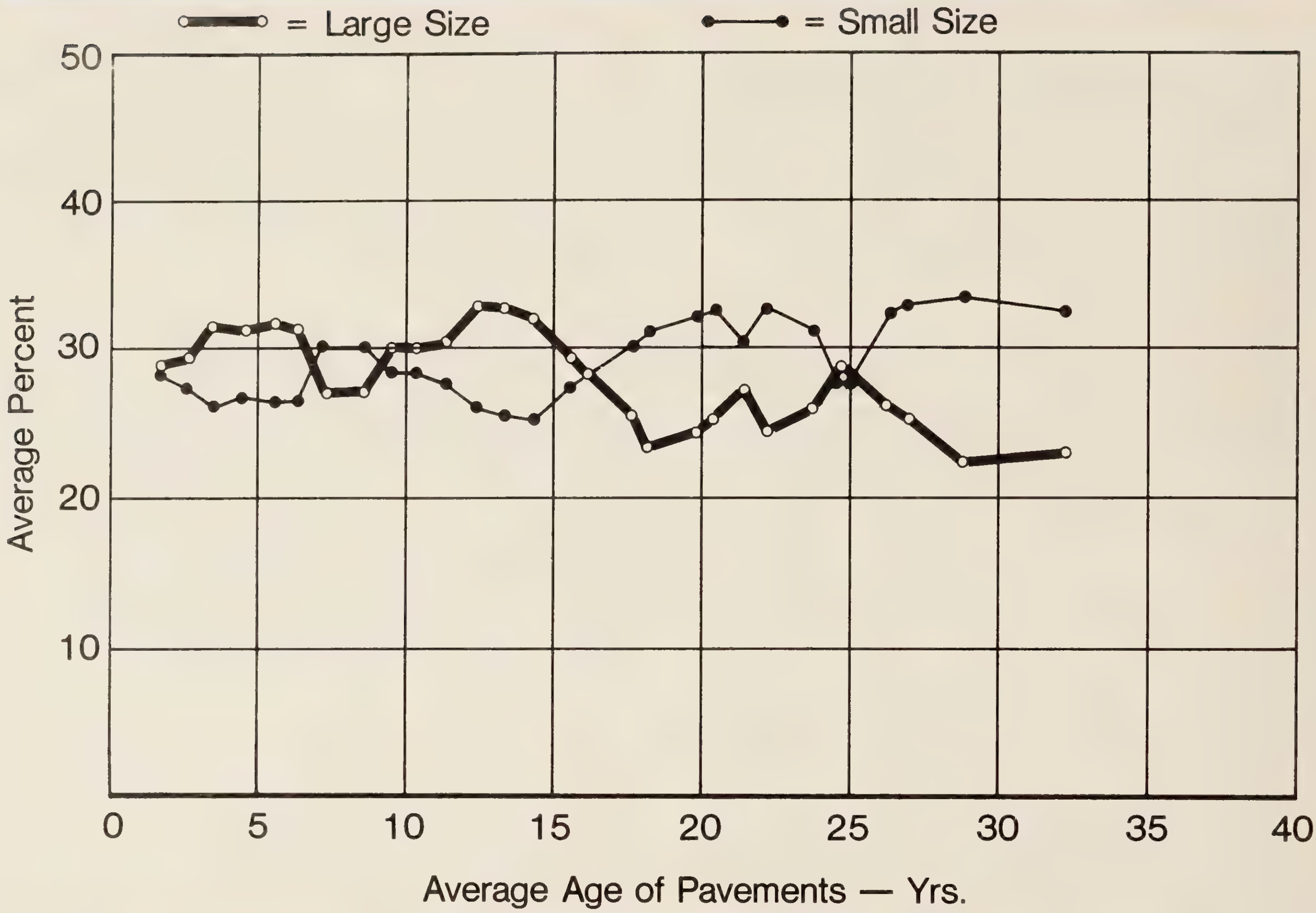

Figure 1.—Average percent large molecule and average percent small molecule content of asphalts from various aged pavements.

Planned Procedure

A systematic approach was chosen to present the computerized STAT derivations and findings of how the large-molecule content and, at times, small-molecule content of the asphalts obtained from 673 pavement samples relate to such factors as condition, rutting, cracking, level of lift, age, traffic, and climate. Each factor is to be presented and discussed separately.

Pavement condition at sampling

Prior to sampling, the pavements that were to provide the asphalt samples for use in this study were rated 1 for excellent, 2 for good, 3 for poor, and 4 for bad. As such coding or rating befits computer use, it was used thus as is evident in tables 2 and 3. According to the contents of tables 2 and 3, no one condition has a mean and dispersion that is significantly different from that of the total sample. Code 2, "good" pavements, comes closest to doing this with its highest average percent of large molecules. The excellent group, code 1, with the lowest group mean percent of large molecules in a group mean comparison with code 2 having the highest, shows a significant difference of means in a t-test of the two groups. Code 2 also has a significant excess mean percent of large molecules when compared by a t-test to the number 4 or "poor" code.

Rutting and asphalt molecular-size content

The t-value obtained for mean and standard deviation when compared to the overall mean and standard deviation shown in tables 2 and 3 indicates that the content of large molecules in the group exhibiting rutting is significantly lower than average. According to the computer, there is only a 3.9 percent possibility that such a departure is due to chance. Table 3 shows that asphalts from rutting pavements have a slightly higher mean level of small molecules. The asphalts from the no-rutting pavements do not differ significantly from average in mean large- or small-molecule content.

Table 2.—A t-test comparison of the mean large-size molecule content of asphalts associated with pavement characteristics or factors when compared with the mean large-size molecule content of asphalts from all pavements in the study

Large Size Molecules:

Characteristics or Factor	Number Reporting	Mean %	Standard Deviation	t-Test value	% Chance Occurrence
Large	673	29.547	7.274		
No Rutting	567	29.824	2.064	+0.680	50.4
Rutting	97	27.902	8.243	−2.046*	3.9
Transverse Cracking	428	29.542	7.423	−0.011	93.9
Longitudinal Cracking	144	29.768	7.656	+0.328	74.0
Random Cracking	109	31.972	6.117	+3.295*	0.1
Alligator Cracking	81	29.755	7.217	+0.260	78.4
Other Cracking	21	27.895	7.505	−1.024	30.7
Condition 1	74	28.743	7.208	−1.317	18.5
Condition 2	102	30.685	6.134	+1.839	6.30
Condition 3	149	30.064	8.238	+0.765	45.1
Condition 4	183	28.907	7.327	−1.055	29.2
Climate 1	102	32.812	5.046	+4.374*	0.01
Climate 2	47	29.194	8.119	−0.320	74.5
Climate 3	40	32.460	5.405	+2.491*	1.3
Climate 4	122	33.775	6.067	+6.048*	0.0
Climate 5	94	25.024	2.752	−5.966*	0.0
Climate 6	83	24.711	6.716	−5.761*	0.0
Climate 7	78	32.677	6.643	+3.628*	0.06
Climate 8	66	27.526	8.763	−2.113	3.3
Climate 9	41	23.871	6.596	−4.875*	0.0
Lift 1	311	29.323	7.159	−0.380	70.4
Lift 2	201	29.446	7.106	−0.113	87.6
Lift 3	128	30.080	7.480	+0.808	42.5
Lift 4	29	29.479	8.611	−0.023	93.0

* Asterisk entries under t-test indicate significant values.

Table 3.—A t-test evaluation of the average small-size molecule content of asphalts associated with pavement characteristics for factors when compared with the average small-size molecule content of asphalts from all the sampled pavements

Small-Size Molecule:

Characteristics or Factor	Number Reporting	Small Molecule Mean %	Standard Deviation	t-Test value	% Chance Occurrence
Small	673	28.039	6.285		
No Rutting	576	27.880	6.030	−0.454	65.4
Rutting	97	28.983	7.595	+1.346	17.5
Trans. Crack. 1xxxx	428	28.128	6.130	+0.232	80.2
Long. Crack. x1xxx	144	27.700	7.680	−0.567	57.8
Random Crack. xx1xx	109	25.838	4.996	−3.482*	0.09
Allig. Crack. xxx1x	81	27.752	7.269	−0.409	68.5
Other Crack. xxxx1	21	28.433	5.384	+0.284	76.8
Condition 1	174	28.347	6.194	+0.586	56.5
Condition 2	162	27.115	5.067	−1.738	7.9
Condition 3	149	27.766	6.469	−0.476	64.0
Condition 4	183	28.776	7.080	+1.368	16.8
Climate 1	102	25.012	2.485	−4.805*	0.0
Climate 2	47	24.991	5.046	−3.251*	0.2
Climate 3	40	23.117	3.961	−4.893*	0.0
Climate 4	4	24.568	6.662	−5.560*	0.0
Climate 5	74	32.022	2.611	+6.069*	0.0
Climate 6	83	33.619	5.560	+7.724*	0.0
Climate 7	78	25.827	4.592	−3.015*	0.3
Climate 8	66	30.730	6.473	+3.311*	0.1
Climate 9	41	33.637	5.532	+5.572*	0.0
Lift 1	311	28.224	5.700	+0.408	68.6
Lift 2	201	27.760	5.908	−0.587	56.4
Lift 3	128	27.841	7.807	−0.336	73.4
Lift 4	29	29.186	7.509	+0.941	34.9

* Significant t-value when mean is compared to overall mean with a t-test
x A presence or absence in the cracking designate

Cracking and asphalt molecular-size content

Table 2 shows a significant increase in the average content of the large molecules in asphalts from pavements exhibiting random cracking alone or with other types of cracking. Actually, the computer indicates that there is only a 0.1 percent probability of a chance occurrence of such a difference. Table 3 shows that the average small-molecule level for the same group of asphalts is significantly lower than the overall average, with a .09 percent possibility of a chance occurrence of such a difference. None of the other types, taken singly or in combinations of cracking groups, transverse, longitudinal, alligator, or other, but without the random type, related to any significant increase or decrease of large- or small-size molecule content of asphalts compared to the overall.

The foregoing findings are no great surprise. Random cracking appears to be fostered by chemical makeup whereas other cracking can be associated with such contributory agents as climate, traffic type and level considerations, etc.

Level of lift and the large- and small-molecule content of asphalt from each

Up to four lifts were evident for some of the pavements sampled. In examining tables 2 and 3, no significant departures in the level of average large-molecule or small-molecule content could be ascertained. Asphalts from lifts 1, 2, and 4 had average large-molecule contents slightly lower than the overall average and lift 3 asphalt's content was higher. As for the small-molecule content, lifts 1 and 4 were greater and 2 and 3 slightly less than the overall average.

The Level of Traffic, Vehicles Per Day (VPD), and Percent Large-Molecule Content of Asphalt

Table 4 summarizes an application of the t-test to the various average percent of large molecules in asphalts from pavements with certain reported VPDs. For the 174 asphalt samples not showing VPD, their reported percents of large molecule contents were included in the overall average large molecule percent content for the 673 total. The traffic levels 1 through 7 were chosen with an eye toward an equitable distribution of the number reporting VPD.

As shown in table 4, a significant lessening of large-molecule content is obtained for the group with the smallest VPD and a significant increase in large-molecule content is shown in the next smallest (1400–2500) VPD group as well as for the 14000–28200 VPD level. Two of these three are explicable. In the lowest VPD, the traffic is of no concern relative to rutting. Therefore, an asphalt which is less stiff, having less

Table 4.—Level of traffic, VPD and percent large-molecule content with t-test values. Average percent large molecule of asphalts and SD for each level of traffic vs overall average percent of 29.2547 and SD = 7.274 for 673 number reporting

Traffic Code	SD	Average VPD	Number Reporting	Percent Large Mol. X_2	SD	t-Test value	Occurrence Percent Chance	Significance
0	(Not reported)	No report	174					
1	(160-1363)	723	115	27.5861	7.754	−2.646	0.82	Yes
2	(1400−2500)	1439	98	32.3735	7.582	+3.574	0.07	Yes
3	(2600−3900)	3303	98	29.5898	7.485	+0.054	91.10	No
4	(4000−6950)	5354	108	28.1714	8.041	−1.798	6.89	No
5	(7050−13000)	9875	103	29.1252	6.808	−0.553	58.75	No
6	(14000−28200)	20825	87	31.7046	5.892	+2.655	0.80	Yes
7	(29800−122000)	43183	64	28.7469	4.311	−0.866	39.13	No

VPD − Vehicles per day of traffic

content of large molecules and therefore more crack-proof due to its less oxidizable smaller molecules, would be selected. In the 14000–28200 VPD level, asphalts capable of bearing such a traffic load without rutting are needed, ergo stiffer large-molecule asphalts were selected to accomplish this. For traffic code 2, the significant increase in large-molecule content over the overall average calls for a conjecture on our part: perhaps there is some unduly heavy VPD traffic in effect or a hot climate condition prevalent at these pavement sites.

Climate and the Level of Large- and Small-Molecules in Associated Construction Asphalts

The following consolidation was made for the climatic designations shown and reported in original order to obtain a balanced number reporting for each:

choices, especially grades of construction asphalts selected by highway agencies. This tendency is even more evident when it was noted that significant differences, t-values, are obtainable for all of the nine climate categories when the small-size molecule content is considered in table 3. Here again, the various highway agencies have followed an old race-track adage, "Horses for Courses," and selected proper grades of asphalt to fit the climate and/or daily traffic.

Concerning table 2, a significantly lesser mean value than that of the overall mean small-molecule content is given for the groups of asphalts obtainable from pavements exhibiting random cracking. This fits with the finding in table 1. In brief, rutting appears to be an observed failing in pavements constructed with asphalts containing significantly lower mean contents of large molecules. Random cracking in pavements appears to relate to a significant greater mean of large-molecule content in asphalts from such pavements. Climate

Number Reporting	Climate Designation	Original Designation	States Involved
102	1	3.1 and 3.2	Georgia, Illinois, New Jersey, Pennsylvania
46	2	4.1, 4.2, 5.1, 5.2	New Mexico, Texas
39	3	8 and 9	New Mexico, Pennsylvania
162	4	14.0	Illinois, Ohio, Pennsylvania
94	5	15.0	Colorado, S. Dakota, Wyoming
82	6	16.0	Idaho, New Mexico, Utah, Wyoming, Colorado
78	7	19.0	Illinois, Minnesota, Idaho
66	8	20.0	Arkansas, Minnesota, N. Dakota
42	9	21.1, 23.0	Arkansas, Colorado, Minnesota, Wyoming

Like the original climate ratings, FHWA's ratings progressed in severity from a relatively mild climate (rated 1) to the 9 category. With only one exception, climate 2, the t-value for each of the others in table 2 has been significant. The mean large-size molecule content is either significantly higher or lower than the overall mean large-molecule content. The implication of this is that climate is a motivating and impelling factor in conditions dictate the use of soft asphalts with a lesser content of large molecules or harder asphalts with a greater content of large molecules. T-test results presented in tables 2 and 3 show that such discretions are being pursued by the various highway agencies in their construction practices.

Group Assembly and Discussion of Future Course of Study

After a lengthy discussion, the study group decided that the data for the younger (age to 16 years old) and older pavements (17 years to 45 years of age) be handled separately because of the apparent differences in large- and small-molecule makeup of the asphalts as indicated in figure 1. In addition to the t-test utilizing the chemical composition data, it was decided to base the t-test on the average levels of traffic associated with particular pavement problems or their planned curtailment.

Age of Pavement and Large-Molecule and Small-Molecule Content of Recovered Asphalt and Pavement Traffic

On the basis of the plot of the average large-molecule content and the accompanying small content shown in figure 1, it was decided to study the two dissimilar asphalt age groups—pavement up to and including 16 years, and pavement aged 17 through 45 years.

Tables 5, 6, and 7 deal with the t-tests associated with comparisons of relationships of large- and small-molecule content of the construction asphalts as well as the traffic factor effect with the various evident pavement failings.

Younger pavements (to 16 years of age), exhibiting rutting (table 5) have an average large-molecule content significantly lower than all other pavements in that age group. The average large-molecule content of asphalts from rutting pavements is significantly lower than that obtainable for the no-rutting pavement asphalts. An accompanying significant increase in the average content of small molecules in asphalts from rutting pavements is not evident in table 6, but the tendency toward such is apparent by the near miss from the significant 5 percent level. Referring again to table 5, we find that two groups of pavements, one having transverse cracking and the other random cracking, have asphalts with a mean large-molecule content significantly higher than the mean content of asphalts from all younger pavements. The t-test of the mean small-molecule content in table 6 finds a significant drop of this mean from the overall mean for the younger group of pavements reporting random cracking.

Using traffic motor vehicles per day as the measuring variable in t-tests, we see in table 7 a number of significant decreases in traffic from that of the overall mean appear for pavements showing transverse and longitudinal cracking. Alligator cracking is close to showing a significant decrease from the mean traffic figure. Random cracking is another near miss. However, the latter type appears to relate to an increase in mean traffic. Therefore, transverse and longitudinal cracking appear to relate to significant decreases in daily traffic from the overall average of 576 sampled younger pavements.

Table 6—Computer t-test comparisons of average percent content of small-size molecules in asphalts from 16-year old and younger pavements having rutting or particular types of cracking with the overall average percent content of small molecules of the 576 asphalts in this age group

Pavement Characteristics	Number Reporting	Small Molecule Mean %	Standard Deviation	t-Test value	% Chance Occurrence
Small (all)	576	27.573	6.191		
No Rutting	500	27.363	5.953	+0.564	58.0
Rutting	76	29.953	7.470	−1.780	7.2
Trans. Cracking	348	27.448	6.041	0.300	75.8
Long. Cracking	123	27.594	8.008	−0.033	92.4
Random Cracking	98	25.497	3.157	3.157*	0.2
Alligator Cracking	68	27.419	7.977	0.187	83.0
Others	114	27.529	5.870	0.026	92.8

* Significant t-value

Table 7.—Computer t-test comparison of average daily motor vehicle traffic experienced by pavements age up to 16 years having certain characteristics with the average DMVT of all the up to 16-year pavements

Pavement Characteristics	Number Reporting	Mean DMVT	Standard Deviation	t-Test value	% Chance Occurrence
Traffic (all)	576	10608.210	12687.310		
No Rutting	500	11059.940	12597.83	+0.544	59.3
5uRutting	76	9668.514	12854.47	−0.877	38.5
Trans. Cracking	348	8925.204	11768.25	−2.007*	4.2
Long. Cracking	123	7839.415	8381.173	−2.314*	2.0
Random Cracking	98	13219.860	12911.55	+1.879	5.7
Alligator Cracking	68	7603.912	11765.61	−1.86	6.0
Other	14	9393.00	16752.12	−0.351	72.4

* Significant t-values

Table 5.—Computer t-test comparisons of average percent large - molecule content of asphalts from 16-year old and younger pavements having rutting or various types of cracking with the overall average content large molecules obtained for the 576 asphalts from the pavements in the age group

Asphalts from Pavements Showing	Number Reporting	Average Large Molecule Content %	Standard Deviation %	t-Test value	% Chance Occurrence of t-value	Significant at 5% Level?
All up to 16-yr pavements	576	30.184	7.228	–	–	–
No Rutting	500	30.501	7.020	+0.727	47.5	No
Rutting	76	28.100	8.224	−2.324	1.9	Yes
Transverse Cracking	348	32.879	8.654	+2.088	3.5	Yes
Longitudinal Cracking	123	30.195	7.879	+0.015	93.6	No
Random Cracking	98	32.542	5.886	+3.061	0.3	Yes
Alligator Cracking	68	30.703	7.153	+0.560	58.3	No
Other Cracking	14	28.386	9.200	−0.914	36.4	No

Further Significant Relationships of Daily Motor Vehicle Traffic (DMVT) and Pavement Factors

In the more comprehensive application of the traffic per day (TPD) variable to considerations of pavement problems and responses to climatic conditions (see table 8), significant differences were found by the computer with t-tests of average amounts of daily motor vehicle traffic for the following at a 5 percent maximum level of chance occurrence:

(1) Transverse cracking;
(2) Longitudinal cracking;
(3) Condition No. 1, excellent;
(4) Condition 3, poor,
(5) Climate No. 1;
(6) Climate No. 2;
(7) Climate No. 3;
(8) Climate No. 6;
(9) Climate No. 7;
(10) Climate No. 8;
(11) Climate No. 9.

It appears that traffic is related to transverse and longitudinal cracking. If significantly higher than average, traffic appears to contribute to or reflect an excellent condition and, if significantly lower than average, to a bad condition. Most climate groups show significant plus and minus departures of average daily traffic from the overall average.

Table 8.—A t-test comparison of the average amount of daily motor vehicle traffic on the pavements associated with various failings or influenced by various factors

Pavement Characteristics	Number Reporting	Mean DMVT	Standard Deviation	t-Test value	% Chance Occurrence
Traffic (all)	673	10056.350	13562.380		
No Rutting	576	10069.17	14196.29	+0.016	93.5
Rutting	97	9980.268	9783.037	−0.053	91.1
Transverse Cracking	428	7961.439	10997.23	−2.683*	0.7
Longitudinal Cracking	144	7470.375	7932.931	−2.208*	2.6
Random Cracking	109	12303.68	12597.47	−1.621	10.1
Alligator Cracking	81	7309.704	10950.94	−1.755	7.6
Other Cracking	21	7960.095	13851.69	−0.697	49.3
Condition 1	174	12665.14	18536.650	+2.084*	3.5
Condition 2	162	11608.25	12990.99	+1.318	18.5
Condition 3	145	6827.618	7826.297	−2.804*	0.5
Condition 4	183	9096.465	11520.920	−0.875	38.6
Climate 1	102	20645.10	21959.47	+6.675*	0.0
Climate 2	47	5304.936	2699.771	−2.397*	1.6
Climate 3	40	21530.5	14457.56	+5.179*	0.0
Climate 4	122	9200.246	8444.377	−0.674	50.8
Climate 5	94	12834.05	16210.11	+1.813	6.7
Climate 6	83	7131.145	6106.339	−1.940	5.00
Climate 7	78	3887.731	3687.311	−3.998*	0.2
Climate 8	66	1340.651	1890.108	−5.213*	0.0
Climate 9	41	5832.463	5027.671	−1.985*	4.5
Lift 1	31	8952.955	13293.980	−1.194	23.1
Lift 2	201	10882.69	14335.86	+0.748	46.1
Lift 3	128	10665.55	12854.23	+0.470	64.4
Lift 4	29	12377.76	13968.63	+0.901	37.1

* Asterisk notes significant ± mean departures from overall average (mean)

A Further T-Test Study of the Significant Associations of Group Study Variables and Overt Pavement Failings Within Each Climate Category for the 576 Younger Pavements

As a final exercise, a decision was made to augment the information shown in tables 5, 6, and 7 for the 576 younger pavements. The exercise consisted of t-test comparisons of the averages of the variables, DMVT, percent large molecules, and percent small molecules, associated with each of several pavement failings like rutting, and particular types of cracking within each climate category with the overall average of each of the three variables in the same climate designation. This was done with respect to each of the nine climate designations.

Table 9 presents the results of the within-climate category comparisons. An arrow pointing up indicates a significant larger average value of variable than the overall average for that climate category. An arrow pointing down indicates a significantly lesser average variable than the overall climatic category average of that variable.

For example, for category 1, the average percent of small molecules of the asphalts associated with the eight reports of rutting is significantly greater than the average percent of small molecules of the 100 reports in climate 1. The 45 reports of transverse cracking show a significantly greater average percent of large molecules and a significantly lesser average percent of small molecules than the average percent of these variables for the 100 reports in climate 1. The 13 pavements with random cracking have a significantly greater average DMVT and a significantly lesser average percent large-molecule content than the overall averages of these variables for the 100 reporting in climate 1.

Actually, the most significant t-test indications are obtainable for the three variables in climate 1; the mildest climate category. Severity of climate increases with number and is greatest for climate 9.

Table 9.—Large and small molecules of asphalts from and average daily traffic on pavements aged 0 to 16 years. Significant t-tests of traffic levels and percents large and small molecules in asphalt

Climate Group	1				2				3			
Pavement	100				29				39			
	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %
No Rutting	92	–	–	–	26	–	–	–	30	–	–	–
Rutting	8	–	–	▲	3	▲	–	–	9	–	–	–
Transverse Cracking	45	–	▲	▼	12	–	–	–	20	▲	–	–
Longitudinal Cracking	16	–	–	–	14	–	–	–	5	–	–	–
Random Cracking	13	▲	▼	–	4	–	–	–	3	–	–	–
Alligator Cracking	10	–	–	–	3	–	–	–	0	–	–	–
Other Cracking	1	–	–	–	2	–	–	–	2	–	–	–

Climate Group	4				5				6			
Pavement	116				76				64			
	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %
No Rutting	91	–	–	–	62	–	–	–	53	–	–	–
Rutting	25	–	–	–	14	–	–	–	11	–	▼	▲
Transverse Cracking	74	–	–	–	42	–	–	–	29	–	–	–
Longitudinal Cracking	46	▼	–	–	7	–	–	–	18	–	–	–
Random Cracking	56	–	–	–	3	–	–	–	8	–	▲	–
Alligator Cracking	13	▼	–	▲	4	–	–	–	0	–	–	–
Other Cracking	0	–	–	–	2	–	–	–	0	–	–	–

Climate Group	7				8				9			
Pavement	73				42				37			
	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %	No. Reporting	Traffic/Day	Large Mol. %	Small Mol. %
No Rutting	73	–	–	–	42	–	–	–	31	–	–	–
Rutting	0	–	–	–	0	–	–	–	6	–	–	–
Transverse Cracking	64	–	–	–	37	–	–	–	25	–	–	–
Longitudinal Cracking	8	–	–	–	3	–	–	–	6	▲	–	–
Random Cracking	6	–	–	–	5	–	–	–	0	–	–	–
Alligator Cracking	31	–	–	–	6	–	–	–	0	–	–	–
Other Cracking	3	–	▲	▼	3	–	–	–	0	–	–	–

▲ = A significantly greater average than the overall climate group average.
▼ = A significantly lesser average than the overall climate group average.

Interpretation of the t-test results contained in table 9 relevant to the effect of the level of our study variables on the presence of certain failings in the sampled pavements is presented as follows:

- Rutting in moderate climates 1 and 2 may be attributable to a significant larger-than-average content of small molecules in the asphalts making up such pavements, a significant greater average traffic per day on pavements in these climate groups, or to both.
- Rutting in the more severe climate 6 resulted from the significant greater-than-average content of smaller molecules, and/or significant less-than-average content of large molecules in the pavement asphalts used in this climate group.
- Transverse cracking in climate 1 is attributable to a significantly larger-than-average content of large molecules and/or significantly lower-than-average content of small molecules in the construction asphalts of the affected pavements. In climate 3, it is mainly attributable to the significantly greater-than-average daily traffic on the affected pavements.
- Longitudinal cracking demonstrated in climates 4 and 9 is associated with the significantly lower-than-average daily traffic on pavements exhibiting such in climate 4. For climate 9, such cracking was due to a significantly greater than average daily traffic on pavements in this climate group.
- Random cracking in climate 1 is attributable to a significantly greater VPD than average rather than the significantly lesser large-molecule content than average found in construction asphalts in this group. In climate 6, random cracking is attributable to a significantly greater than the average percent large-molecule content of the paving asphalts in this group.
- Alligator cracking in climate group 4 is attributable to a significant decrease in the VPD on the affected pavements compared to the average traffic of the climate group, with an assist of the significantly higher-than-average level of the average percent of small molecules of affected pavement asphalts compared to that of all the pavement asphalts from climate 4.
- Other cracking in climate 7 is attributable to a greater-than-average content of large molecules and a lower-than-average of small-molecule content of asphalt from afflicted pavements in this group compared to that of all asphalts from pavements in climate 7.

Pavement Factors and Large-Molecule and Small-Molecule Contents of Asphalts from Sampled Pavements and Daily Traffic on Pavements Aged 17 Years or Older

Tables 10 and 11 show no significant differences in average large- or small-molecule content in asphalts from the 87 pavements in this age group when such data is subjected to t-test comparisons similar to that applied to the data from the younger age group. This also applies to t-tests applied to mean daily traffic comparisons (see table 12) of pavement failings including rutting and transverse, longitudinal, random, alligator, or other cracking.

Summary

The following have been the findings of this statistical and chemical study:

(1) Asphalts sampled from pavements that were constructed 17 or more years ago are different from asphalts contained in younger pavements. The difference is in the mean content of large and small molecules; the older series of pavements yields asphalts containing a greater mean percent of small molecules and the newer construction yields asphalts with a larger mean content of large molecules. Of course, pavements constructed more than 16 years ago with asphalts having a larger content of larger molecules did not last, and we have only survivors originally built with asphalts having greater amounts of small-molecule contents.

(2) Relevant to condition, t-test comparisons of condition relationships with molecular size content of the asphalts from the various rated pavements were not distinctly informative. Other factors seemed to be involved relevant to condition.

(3) Rutting pavement asphalts have a significantly lower content of large molecules than average. The average content of small molecules of these asphalts do not show significant differences from average. Stiffer or harder asphalts with a sufficient level of large molecules can be used to prevent this distress.

(4) T-test of the molecule sizes of asphalts from all sampled pavements attributes significant consideration to

Table 10.—Computer t-test comparisons of average percent large molecule content of asphalts from 17 through 45-year old pavements having rutting or various types of cracking with the overall average content of large molecules obtained for the 87 asphalts from pavements in this age group

Asphalts Pavements Showing	Number Reporting	Average Large Molecule Content %	Standard Deviation	t-Test value	Chance Occurrence of Such a Different %	Significant at 5% Level?
All 17 to 45-Year Pavements	87	25.489	6.590			
No Rutting	69	25.026	5.734	−0.461	65.0	No
Rutting	18	27.261	9.165	+0.967	33.8	No
Transverse Cracking	72	25.351	5.968	−0.136	86.2	No
Longitudinal Cracking	19	26.579	5.579	+0.670	51.1	No
Random Cracking	11	26.891	6.219	+0.669	51.2	No
Alligator Cracking	13	26.846	5.501	+0.706	48.9	No
Other Cracking	7	26.914	2.422	+0.567	57.9	No

Table 11.—Computer t-test comparison of the mean percent of small molecule content in asphalt from pavements with particular characteristics in the age 17 to 45 category with the overall mean content for this group

Pavement Characteristics	Number Reporting	Small Molecule Content Mean	Standard Deviation	t-Test value	% Chance Occurrence
% Small Molecule	87	30.851	6.359		
No Rutting	69	31.390	5.488	−0.559	58.4
Rutting	18	28.783	8.852	1.168	24.4
Transverse Cracking	72	30.957	5.910	−0.108	87.8
Longitudinal Cracking	19	28.853	5.487	1.269	20.5
Random Cracking	11	28.873	5.418	0.986	32.8
Alligator Cracking	13	28.608	4.434	1.225	22.1
Other Cracking	7	30.243	4.028	0.248	79.2

Table 12.—Computer t-test comparison of mean daily motor vehicle traffic on pavements age 17 to 45, with certain characteristics with the overall mean daily motor vehicle traffic for the 87 pavements in the 17 to 45-year old group

Pavement Characteristics	Number Reporting	Mean DMVT	Standard Deviation	t-Test value	% Chance Occurrence
Traffic	87	7099.712	18593.48		
No Rutting	69	7032.319	20658.57	0.021	93.2
Rutting	18	7358.056	6459.826	−0.058	90.9
Transverse Cracking	72	3811.181	5109.835	1.455	14.4
Longitudinal Cracking	19	5197.	4162.799	0.443	66.3
Random Cracking	11	4141.364	3232.077	0.524	60.8
Alligator Cracking	13	5770.769	4851.057	0.255	78.7
Other Cracking	7	5094.286	4141.424	0.283	76.9

the higher-than-average level of large molecules and the lower average level of small molecules as related to random cracking. No other types of cracking were associated similarly. Random cracking is the most logical type attributable to chemical considerations of the asphalts involved in such pavements. Other cracking may be related to the size of molecules in the pavement asphalt, but it is obvious from the t-test findings that their source lies with some other factor or factors, e.g., climate, traffic load and volume, and construction.

(5) In younger pavements, transverse and random cracking are linked significantly to a larger-than-average content of large molecules in asphalts recovered from such pavements. Rutting in this group relates to a significantly lower-than-average content of large molecules in the asphalts recovered.

(6) In younger pavements, transverse and longitudinal cracking are linked significantly to vehicle level per day.

(7) Pavement condition appears to rely on level of traffic. A significant increase in average daily traffic compared to an overall average contributes toward excellent condition pavements, but a significant decrease from average ties in with poor pavements. This may be placing the cart before the horse in cause and effect and that condition is actually affecting the daily traffic level.

(8) In older pavements, no rutting or cracking problems can be significantly attributed to the large- or small-molecule content of their asphalts or to the level of traffic experienced.

Conclusion

It appears that High Performance Liquid Chromatography (HPLC) determinations of the large- and small-molecule content of asphalt can be an acceptable approach for quality control, especially when developed with an acceptable precision. Not only would chemical constancy of asphalts be maintained for particular crude sources, it would also help in the selection of asphalts derived from various combinations of crude. Additionally, it also may be useful for monitoring the actual refining or asphalt production process.

The prime applications of such determinations would be incorporation into specifications of percentages of large and low molecules for purchase of paving asphalts for particular climates or traffic conditions in order to prevent rutting or certain types of cracking.

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Retroreflective Requirements for Traffic Signs— A Stop Sign Case Study[1]

by

Juan M. Morales

Introduction

Traffic signs are vital to driver safety. They are, however, only effective inasmuch as they have good recognition distances—both in the daytime and at night. The *Manual on Uniform Traffic Control Devices* (MUTCD) mandates that regulatory and warning signs be reflectorized or illuminated to show the same shape and color by day or night. While the MUTCD contains standards for size, shape, and color, it provides no minimum requirements for traffic sign retroreflectivity. (*1*)[2] The absence of such initial and inservice requirements means there are no national standards for replacement of those signs lacking appropriate retroreflectivity.

Signs come in various designs and shapes to serve different needs. These size and shape differences create substantial corresponding differences in the retroreflectivity needed to function properly at night. Thus, different signs can perform adequately with different levels of retroreflectivity.

This article summarizes a study on Stop sign performance as a function of retroreflectivity. (*2*) It details a procedure for assessing sign adequacy based on retroreflective properties—i.e., a method for determining if a particular sign needs to be replaced. Stop signs were selected for the study since they are critical and must have adequate retroreflectivity at night to convey their intended message and avoid potential accidents with injuries or even fatalities.

Study findings will:

- Aid in establishing minimum inservice retroreflectivity levels for Stop signs.

[1] Mr. Morales was awarded the Institute of Transportation Engineers Past President's Award for his contribution to the study on which this article is based. An article similar to this one was published in the November 1987 *ITE Journal*.

[2] Italic numbers in parentheses identify references on page 89.

- Assist field personnel in determining if a Stop sign provides the desired recognition distance or if it should be replaced.
- Provide insight for creating minimum retroreflective standards for other sign types.

Approach

To perform the study, the project team first obtained a set of 30-in (76 cm) Stop signs with a wide range of retroreflective characteristics. These signs were laboratory-tested to measure their retroreflective properties. Field recognition distance was then determined for a selected subset. Using regression analysis techniques, retroreflectivity versus recognition distance relationships were developed. Based on statistical goodness of fit and other criteria, the best relationship was selected. From this relationship, and considering the approach speed of the intersections, a procedure to determine when Stop signs should be replaced was developed. This approach is detailed below.

Laboratory Procedure

Terminology

Retroreflectivity refers to the fraction of light that returns to the eye after striking a surface. This is commonly, and somewhat erroneously, referred to as "brightness." Erroneous, since brightness—as it refers to the intensity of the *sensation* resulting from viewing surfaces from which light comes to the eye—is not a measurable quantity. Retroreflectivity, on the other hand, can be measured in terms of the luminance (L) and the specific intensity per unit of area (SIA) of the surface.

Luminance (photometric brightness) is the luminous intensity of a surface in a given direction per unit of projected area of the surface as viewed from that direction. When the luminous intensity is in candelas and the area is expressed in square feet, luminance is in candelas per square foot (cd/sq ft).

Specific intensity per unit area is the ratio of the luminous intensity of the surface to the normal illuminance and to the area of the retroreflective surface. It is expressed in candelas per footcandle per square foot (cd/fc/sq ft).

Both L and SIA are affected by the observation angle (the angle between the line from the light source to the sign and the line from the driver's eye to the sign) and are not a property of surface material. Changes in the observation angle will result in changes to both L and SIA.

Study sample and laboratory design

In the summer of 1985, Washington, DC, replaced some 3,100 Stop signs. Although the average life expectancy of a sign is 7 years, some of these signs had been in service for 20. The study team contacted city traffic officials and from them obtained approximately 50 Stop signs from which a set of 35 Stop signs was selected for study use. Signs chosen ranged from "good" to "badly faded." Included in the set were a new engineering grade sign and a new high intensity sign. Two high intensity signs were included for a relative comparison.

A photometric laboratory was prepared at the Turner-Fairbank Highway Research Center in McLean, Virginia. The laboratory consisted of a 100-ft (30.4 m) black tunnel, baffles, headlights, a headlight stand, sign stand, and various photometric instruments. Baffles were used to minimize the light reflecting off the tunnel walls, floor, and other surfaces illuminating the test signs.

Signs were illuminated with automotive headlights located 100 ft (30.4 m) away and their reflective properties measured from a fixed observation point. Two standard 12.8 V automotive headlights, mounted on an adjustable stand, were used to illuminate the test signs. The headlights and signs were in the same horizontal plane. Angular offsets were used to orient the headlights relative to the signs to simulate signs in a mounting position of 7-ft (2.1 m) high and 2 ft (0.6 m) off the pavement's edge.

Data collection

Once the laboratory was completed, the photometric data were collected as follows:

- Luminance was measured using a Pritchard 1920 Telephotometer set at a 2-minute aperture. Eight points in the red area of the sign and eight points in the white area (letters only) were read. From this, both a "red" average and a "white" average were obtained. These averages were weighed by the respective percentage of red (76 %) and white (24 %) in the sign to obtain an "overall sign luminance."
- Specific intensity per unit area was measured using a Retro-Tech 920 Meter. This gun-like instrument is calibrated for different colors and materials and is pressed against a point on the test surface. Using the same red (76 %) and white (24 %) areas, the averages of the eight "red" SIA readings and eight "white" SIA readings were weighed to obtain an "overall sign SIA." The Retro-Tech 920 has its own built-in light source, which strikes the test surface at a 0.2-degree-observation angle. As noted above, variations in the observation angle will result in significant variations in the measured SIA.
- White to red ratio (W/R), or internal contrast, was defined as:

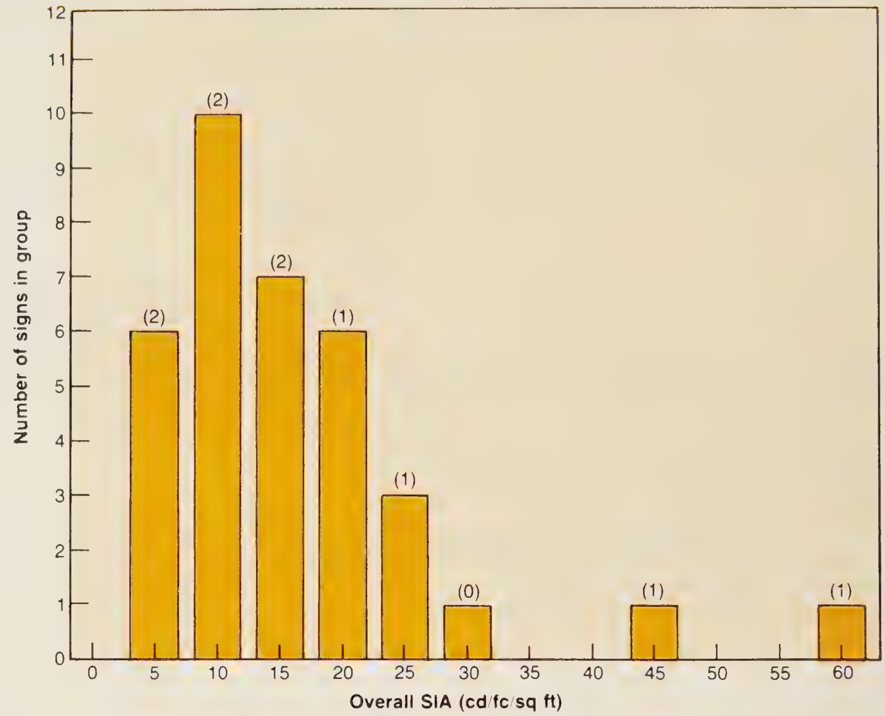
$$W/R = \frac{\text{White SIA}}{\text{Red SIA}}$$

Figure 1 shows the sample distribution obtained from the laboratory data for the overall SIA measure. Based on this distribution, a subset of 10 signs was chosen for field testing. Figure 2 shows 9 of the 10 selected signs, arranged from left to right in order of increasing overall SIA, during daylight and nighttime conditions. This figure clearly shows how signs seeming to be adequate during the day, might not perform adequately during the night.

Field Procedure

Of all the components of the perception-reaction process involving a driver approaching a sign, the one of interest to this study was the point of recognition. This is the point at which the driver recognizes and understands the command associated with the sign being approached. The distance from this point to the sign is called "recognition distance." This distance can be equal to or longer than the "legibility distance" (i.e., the distance at which the words on a sign can actually be read).

For a Stop sign, most drivers recognize the command associated with the familiar red octagon before they are sufficiently close to read the word Stop, if they read it at all. The message is conveyed by code (color, shape, etc.), not only by legibility. This is borne out by a 1957 study done by Faber Birren. (*3*) In this study, a conventional Stop sign was placed in a prominent location with the letters rearranged to read "TOPS." Of the 100 drivers questioned after passing the sign, 86 percent admitted that they had not noticed the misspelling, indicating that the sign is primarily recognized by its shape and color.



( ) Indicates number of signs selected for field testing

Figure 1.—Sample distribution.

Sign No. Overall SIA (cd/fc/sq ft)					
	14 (2.4)	7 (3.1)	31 (5.7)	6 (7.6)	28 (11.5)
	30 (13.9)	25 (17.7)	1 (24.1)	33 (42.2)	

@Kodak Ektachrome 64 film, f5.6, 1 sec. exposure
*Sign No. 35 not shown

Figure 2.—Field-tested signs during daylight and nighttime conditions.

In the present study, 20 paid subjects were selected to drive down a controlled road to determine the recognition distance of the selected signs. The road was 2,000-ft (610 m) long, level, totally dark, and had no other traffic. The subjects chosen were a cross section of the driving population in terms of percentage of vehicle miles driven annually. Two age groups (under 35 and over 35) were chosen; half of the subjects were male and half female to represent the driving population.

Sign recognition distance was measured using a calibrated fifth-wheel mounted to a Ford Tempo sedan equipped with aligned Wagner H6054 low-beam headlights. Subjects were instructed to, drive toward the signs (7-ft (2.1 m) high, 2 ft (0.6 m) off pavement's edge), one at a time, and to say aloud when, without any doubt, a sign was recognized. The fifth-wheel counter was then immediately reset to zero by the observer in the passenger's seat, and the subject continued to drive up to the sign. The distance measured between the reset point and the sign location was defined as the sign's recognition distance. Upon passing the sign, the subject made a "U" turn to return to the starting position. An assistant then replaced the sign for another at the same location; this procedure was repeated for each of 10 Stop signs and 5 "dummy" signs used to eliminate Stop sign expectancy. Subjects were not aware that the study pertained exclusively to Stop signs.

Under average driving conditions there are countless distractions —pedestrians, other traffic, and wandering eye and mind. Results from the procedure used in this study provide recognition distances under ideal conditions, i.e., a dark background on a clear night. The effects of background complexity, weather conditions, and environmental distractions were not considered in this study.

Table 1 summarizes the collected photometric and recognition distance data for the 10 signs selected for field testing.

Data analysis

To develop a field procedure for assessing adequacy of sign retroreflectivity, the study team next attempted to design a Stop sign photometric-recognition distance model for nighttime application.

Preliminary analysis

Figure 3 illustrates the relationship among subject response, recognition distance, and SIA values. As the overall sign SIA increases (moving to the right), so does the number of subjects who could recognize the

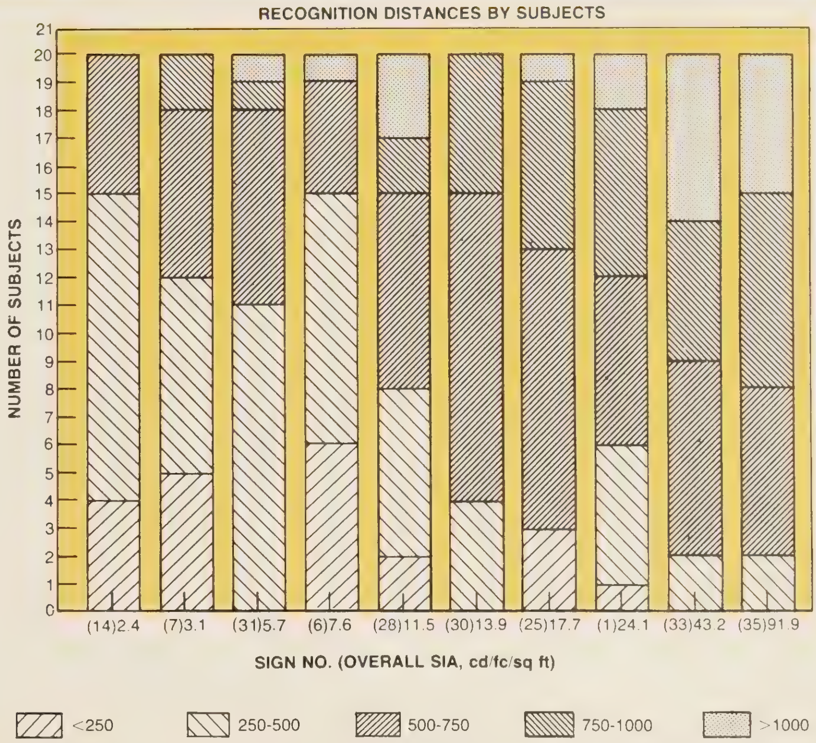


Figure 3.—Frequency counts.

Table 1.—Collected data for signs tested in the field

	LUMINANCE, CD/SQ FT				SIA, CD/SQ FT/CF			RECOGNITION DISTANCE (IN FEET)				
SIGN NO.	OVERALL	WHITE	RED	W/R RATIO	OVERALL	WHITE	RED	MIN.	85%	AVG	MAX	STD
14	0.02	0.04	0.01	2.5	2.4	4.5	1.8	210	250	431	710	146
7	0.02	0.00	0.03	0.4	3.1	1.6	3.6	170	240	443	860	182
31	0.06	0.22	0.01	11.9	5.7	19.0	1.6	345	360	545	1130	198
6	0.06	0.04	0.07	0.5	7.6	4.5	8.6	128	175	374	1040	218
28	0.09	0.09	0.09	0.7	11.5	9.2	12.3	150	300	645	1580	377
30	0.14	0.43	0.05	5.8	13.9	37.8	6.5	372	467	650	980	185
25	0.19	0.46	0.10	3.8	17.7	40.7	10.6	320	500	682	1020	201
1	0.29	1.08	0.04	18.0	24.1	86.2	4.8	120	350	672	1400	290
33	0.48	1.59	0.14	10.6	43.2	139.8	13.2	352	533	879	1650	345
35	0.98	3.19	0.30	10.4	91.9	296.4	28.4	405	525	869	1675	338
MIN: 0.02		0.00	0.01	0.4	2.4	1.6	1.6	120	175	374	710	166
MAX: 0.98		3.19	0.30	18.0	91.9	296.4	29.4	405	533	879	1775	377

sign at a longer distance. For example, 15 of the 20 subjects (75 %) had to be within 500 ft (150 m) of Stop sign 14, which had the lowest SIA value, to provide correct recognition. For sign 35 however, with the highest SIA value, only 8 of the 20 subjects (40 %) had to be within 500 ft (150 m) of the sign. This proportionality confirmed that a relationship existed between the photometric characteristics of a sign and its recognition distance.

Modeling of inservice Stop sign night recognition distance

A mathematical model was sought which related both retroreflectivity values and nighttime Stop sign recognition distance. Preliminary examination of the data suggested that the Box-Hunter equation of the form $y = a[1-e^{-bx}]$ would be suitable In this model, y is the recognition distance, x is the photometric measure (SIA, for example), a and b are regression coefficients, and e is the exponential constant 2.71828. This exponential model assumes that at zero L or SIA, the recognition distance is also zero, and that beyond certain distances, regardless of the sign's photometric brightness, the sign cannot be recognized.

Nonlinear regression analysis techniques were used to develop Box-Hunter models for recognition distance versus each of the following photometric quantities: (*4, 5*)

- Red luminance.
- White luminance.
- Overall luminance.
- Red SIA.
- White SIA.
- Overall SIA.

All 200 data points (20 subjects × 10 signs) were used to generate the a and b regression coefficients for the six mathematical models. The models generated using this procedure represent the average, or the 50th percentile, of the 200 data points.

Best model selection

The next goal of the study was to develop a field procedure using the best of the six photometric-recognition distance models. Runs and Durbin-Watson tests were conducted to determine the goodness of fit of the nonlinear regression analysis. These tests, and an F-test approximation found no statistical evidence at the 0.05 significance level that any of the six models did not represent the observed data. (*5*) Table 2 contains these test results, as well as the a and b coefficients generated for each model.

Based on these results, the "overall SIA" model was selected as the best model. Besides its statistical goodness, it explicitly accounts for the retroreflectivity of the entire sign and, therefore, the retroreflectivity of both the red and white, regardless of material (engineering grade or high-intensity sheeting). In addition, it can easily be measured in the field during daylight hours using an instrument such as the Retro-Tech 920, which has its own light source at a constant observation angle. Determining the observation angle in the field (which would be necessary if a luminance-based model were selected) is not necessary.

The selected photometric-recognition distance model is:

$$\text{30-in (76 cm) Stop Sign Recognition Distance, in feet} = 762(1-e^{-.19(SIA)})$$

where SIA is the overall sign SIA (.76 Red SIA + .24 White SIA) in candelas per footcandle per square foot.

As noted above, this model represents the 50th percentile of the 200 data points analyzed. Common traffic engineering practices use the 85th percentile for design purposes. To facilitate computations, the 85th percentile of the recognition distance of the 20 subjects was computed for each sign. Regression analysis was then performed on the resulting 10 points to obtain the Box-Hunter 85th percentile model:

$$\text{30-in Stop Sign Recognition Distance, in feet} = 476(1-e^{-.15(SIA)})$$

where SIA is the overall sign SIA, in candelas per footcandle per square foot. This model satisfies 85 percent as opposed to 50 percent of the test subjects. Both the 50th percentile and the 85th percentile models are shown in figure 4.

Table 2.—Regression Analysis Results (for the "50th percentile")

Box-Hunter equation: $y=a[1-e^{-bx}]$

y	x	Coefficient a	Coefficient b	Runs Test: Runs	Runs Test: Z-Stat.	Durbin-Watson Stat.	Residual Sum of Squares (3)
30-inch Stop Sign Recognition Distance in feet, under ideal visibility conditions	Overall SIA (1)	762	0.19	73	−3.86	1.53	2.42
	Red SIA (1)	683	0.57	77	−3.24	1.39	2.47
	White SIA (1)	715	0.24	71	−4.17	1.47	2.41
	Overall Luminance (2)	751	2.06	77	−3.36	1.49	2.42
	Red Luminance (2)	683	11.7	77	−3.15	1.37	2.46
	White Luminance (2)	725	1.66	71	−4.25	1.27	2.43

(1) in candelas per footcandle per square foot
(2) in candelas per square foot
(3) in square feet $\times 10^7$

From the overall SIA model obtained for 30-in (76 cm) Stop signs, data for 24-in (61 cm) Stop signs can be derived, assuming that, if the shape, color, and overall SIA of a 30-in Stop sign is equal to those of a 24-in (61 cm) Stop sign, the difference in relative recognition distance is directly proportional to sign area. In other words, if all factors remain constant except size, recognition distance can be assumed to be directly proportional to the area of the sign. (*6*)

The area of a 24-in (61 cm) Stop sign is 64 percent of the area of a 30-in (76 cm) sign; therefore, its recognition distance will be 64 percent of the distance obtained from the mathematical model. Table 3 shows the 50th and 85th percentile data for 30-in and 24-in Stop signs for a given overall SIA. Note that a higher than 40 overall SIA does not improve on the sign's recognition distance.

Table 3.—Stop sign recognition distance versus overall SIA under ideal visibility conditions (Selected model in tabular form)

Overall SIA cd/fc/sq ft	STOP Sign Recognition Distance, Ft			
	30-in		24-in	
	50th %	85th %	50th %	85th %
0	0	0	0	0
1	129	68	83	44
2	237	126	151	81
3	326	176	208	113
4	400	219	256	140
5	461	256	295	164
6	512	288	328	184
7	555	315	355	201
8	590	338	377	216
9	619	358	396	229
10	643	375	412	240
11	664	389	425	249
12	680	402	435	257
13	694	412	444	264
14	706	422	452	270
15	715	430	458	275
16	723	436	463	279
17	730	442	467	283
18	735	447	471	286
19	740	451	474	289
20	744	455	476	291
21	747	458	478	293
22	750	461	480	295
23	752	463	481	296
24	754	465	482	298
25	755	467	483	299
26	756	468	484	300
27	757	469	485	300
28	758	470	485	301
29	759	471	486	302
30	760	472	486	302
31	760	473	486	303
32	760	473	487	303
33	761	474	487	303
34	761	474	487	304
35	761	475	487	304
36	761	475	487	304
37	762	475	487	304
38	762	476	488	304
39	762	476	488	304
40	762	476	488	305

Field application of model

The 85th percentile overall SIA model presented in table 3 can be adapted to a field procedure for assessing whether an inservice Stop sign has sufficient retroreflectivity or should be replaced with a new sign. This procedure applies the standard stopping distance formula to table 3. If the distance required by a driver to recognize the sign and come to a complete stop at a comfortable deceleration is known, then the overall SIA needed to achieve this distance can be obtained from the model selected above for a given approach speed.

For example, using a 1.5-second perception-reaction time and a comfortable deceleration of 8 ft/s/s, for an approach speed of 35 mi/h (56 km/h) or 51.3 ft/s (15.6 m/sec) a minimum viewing distance of

$$t(V) + (V^2/2a)$$

or

$$1.5(51.3) + (51.3^2/16) = 242 \text{ ft } (74\,\text{m})$$

is needed. From table 3, for the 85th percentile column, the 30-in (76 cm) Stop sign overall SIA (under ideal viewing conditions) needed to provide at least 242 ft (74 m) of stopping distance is 5 cd/fc/sq ft.

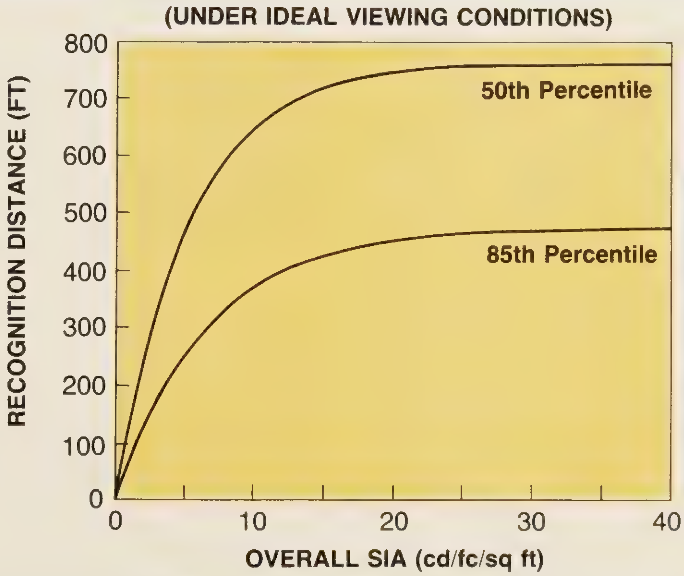


Figure 4.—30-in (76 cm) Stop sign (under ideal viewing conditions).

Using this procedure, the required overall SIA for various approach speeds can be obtained as shown in table 4. For example, for an intersection with a 35 mi/h (56 km/h) approach speed, a 30-in (76 cm) Stop sign with a minimum overall SIA of 5 would be sufficient to accommodate the 85th percentile of drivers. Higher values should be used to accommodate less than ideal viewing conditions.

To implement the procedure in the field, a user would perform the following steps:

(1) Measure the sign's red SIA and white SIA (in cd/fc/sq ft), using a retroreflectometer such as the Retro-Tech 920.

(2) Multiply the red SIA by .76 (or 3/4).

(3) Multiply the white SIA by .24 (or 1/4).

(4) Add the results of steps 2 and 3 together to obtain the overall sign SIA.

(5) Determine the average approach speed of the intersection.

(6) Obtain the minimum overall SIA from the appropriate column in table 4.

(7) Replace the sign if the computed overall SIA (step 4) is less than the overall SIA obtained from table 4 (step 6).

When applying this procedure, the user should consider using higher values of overall SIA to accommodate less than ideal viewing conditions resulting from inclement weather, glare, etc. Stop Ahead signs should be used when horizontal or vertical sight distance restrictions are present.

Table 4.—Required overall SIA for various approach speeds under ideal visibility conditions

INTERSECTION'S APPROACH SPEED		STOPPING DISTANCE*	MINIMUM OVERALL SIA** (cd/fc/sq ft) 85th Percentile	
MI/H	FT/S	FT	30-in STOP	24-in STOP
10	15	35	1	1
15	22	63	1	2
20	29	98	2	3
25	37	139	3	4
30	44	187	4	7
35	51	242	5	11
40	59	303	7	35
45	66	371	10	40
50	73	446	18	40
55	81	528	40	40
60	88	616	40	40
65	95	711	40	40

* Assumes a deceleration of 8 ft/s/s and a perception-reaction time of 1.5 seconds

** Under ideal visibility conditions (does not consider weather nor other visibility reducing factors) Stop sign's overall SIA in candelas per footcandle per square foot, = [.76 × Red SIA] + [.24 × White SIA]

Discussion

The recommended model and field application procedure described here can be used as an indicator to assess the adequacy of Stop signs based on their retroreflective properties to satisfy driver needs during nighttime conditions.

The model does not indicate the adequacy of Stop signs for daytime driving in which, in accordance with the MUTCD, Stop signs should have white letters on a red octagonal background.

Lack of color should not be confused with lack of retroreflectivity. For example, a Stop sign could lose red pigment to the point where it does not comply with the MUTCD color criterion, and still have sufficient retroreflectivity. (Such a sign, however, should be immediately replaced, regardless of its retroreflectivity, since it does not meet MUTCD criteria.)

In order to screen cases such as the one described above, consideration was given to include a second parameter, such as white to red (W/R) ratio. Previous research studies have found the optimum W/R ratio to be between 8 and 12. (*7, 8*) Inclusion of a contrast-based parameter would not only complicate the field-screening procedure, but could cause the discarding of hundreds of signs because they are not "legible." Internal contrast, or W/R ratio, becomes important when the sign in question must be read to be understood. In the case of the uniquely shaped Stop sign, the sign does *not* have to be read to be understood and, therefore, inclusion of a contrast-based screening parameter would eliminate those signs which are perfectly recognizable but not legible; legibility should not be a criterion for developing inservice retroreflective standards for Stop signs.

Furthermore, additional research would be needed to determine the acceptable band of W/R ratios. Should it be between 8 and 12 or, for example, between 3 and 25? Considering the range of W/R ratios of the field-tested signs (from 0.4 to 18.0) gives an idea of the implication of adding a second screening parameter with various bandwidths.

Such questions remain unanswered and must be considered before inservice retroreflective standards are established. This study, has confirmed the need for such standards, and the results will help quantify the adequacy of Stop signs for nighttime recognition. Further research, however, is necessary to determine minimum levels of retroreflectivity of other sign types and to improve on ways of measuring retroreflectivity in the field.

Summary

This study found mathematical relationships between Stop sign recognition distances and the signs' photometric characteristics. It has also recommended using a recognition distance versus the overall SIA model for field applications to determine when Stop signs should be replaced based on the overall SIA of the sign and the intersection's approach speed.

While this study has contributed to the understanding of inservice sign retroreflectivity, other research is underway to further investigate the problem. The Federal Highway Administration is conducting research to determine the minimum distances at which signs and markings should be visible to a motorist. Based on these minimum visibility requirements, it will be possible to determine the retroreflectivity necessary to make a sign or marking visible at a given distance. This study is expected to be completed in late 1988.

A National Cooperative Highway Research Program study will determine the feasibility of developing instrumentation suitable to rapidly measure retroreflectivity from a moving vehicle during daylight hours. This study is expected to be completed in early 1989.

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MODELING TRAFFIC FLOW

by
Paul Ross

Introduction

The only practical way at present to analyze short-term fluctuations of traffic is by simulating the motion of individual vehicles. Such "microscopic" analyses of traffic work very well but are impractical for on-line control or simulation of traffic networks larger than just a few city blocks. For practical understanding of real-size traffic systems and traffic control, an accurate "macroscopic" representation of traffic is required. However, given the broad range of traffic behavior patterns, effectively modeling traffic flow is a difficult, complex process.

Recently, however, a new traffic model has been developed. In this formulation, traffic is modeled as a compressible fluid with a limit to its compressibility, and formulation is given in terms of partial differential equations.

This new traffic flow model is far superior to all previous macroscopic representations of traffic. It is well suited for both traffic simulation—allowing faster and more accurate programs than are currently available—and for traffic control.

Defining and Correlating the Variables

It is generally agreed that the most important traffic variables are traffic volume, density, and average speed. Volume, the number of vehicles passing a point per unit time, is usually given in vehicles per hour (veh/hr), even if the observation was made for only 5 minutes. Traffic density is the number of vehicles per unit length of road, usually given in vehicles per mile (veh/mi). Average speed is simply the average of the speeds of all vehicles on the length of road.

There are, however, two possible ways to measure this average traffic speed. One way is to measure the speeds of all vehicles that pass a certain point, for example, during an hour and average them. Since the observations are distributed over time at one spot, this is called the "time mean speed" or "spot speed." The second way is to consider all vehicles located on a certain mile of road at a given instant and take the average of their speeds. This is called the "space mean speed." The space mean speed is the more significant measurement; it is what will be meant whenever the word "speed" is used in this article. (Research, however, has shown that for many purposes the two speeds are the same.) (*1*)[1]

What is needed is a set of relationships such that, if the traffic volume, density, and speed are known everywhere at some starting time; and if those variables for the traffic entering the road (or traffic network) are known, the volume, density, and speed at any place in the network at any time can be calculated.

Since there are three traffic variables (volume, density, and speed) three relationships must be established among them. In determining these relationships, it is useful to think of traffic as a fluid. From this premise, two relationships at least can be easily defined. The first relationship among volume, density, and speed is inherent in the quantity definition:

$$Q = k \cdot v \qquad (1)$$

where

Q = traffic volume (veh/hr) past the point,
k = vehicular density (veh/mi), and
v = (local space-mean) speed (mi/h)

A second relationship, that vehicles are not created or destroyed while on the road, was pointed out by Lighthill and Whitham: (*2*)

$$\delta k/\delta t + \delta Q/\delta x = S(x,t) \qquad (2)$$

where

$\delta/\delta t$ and $\delta/\delta x$ indicate partial differentiation with respect to time t and with respect to location along the road x, respectively; and
$S(x,t)$ = source strength of vehicles from ramps (negative for off-ramps) (veh/mi/h).

Both of these equations are true for all fluids, including traffic. It is the third (yet to be determined) relationship that differentiates one fluid from another. For example, water and other incompressible fluids have a third equation reflecting their incompressibility; similarly, the third equation-of-state for compressible fluids (e.g., gases) generally introduces new variables such as temperature (which then require additional equations). Here, a third equation-of-state needs to be determined for traffic; further, the goal is to not introduce any new variables requiring still more equations of state.

Deterministic Speed-Density Hypothesis

The earliest proposal for the third equation dates from 1934. Bruce Greenshields reported that speed on rural highways depends upon traffic density and that the speed-density curve is approximately a straight line with speed decreasing as density increases. (*3*)

Subsequent researchers have assumed that there is a well-defined, deterministic relationship between traffic speed and density; they have not, however, always agreed on the exact shape of the relationship. (See reference 4 for a summary of speed-density hypotheses and supporting data.)

The idea that there is a deterministic relationship between speed and density, be it straight line or curve, is simply untenable. The most obvious problem is that speed-density observations always have much more scatter than can be explained by any reasonable amount of experimental error. Nevertheless, there are a few papers published every year showing the "speed-density curve" for some location or road type, usually with the embarrassingly scattered data points omitted and a single curve shown.

"Desired" Speed-Density Hypothesis

Prigogine and Herman, in a series of papers starting in 1961, described traffic speeds as relaxing to a "desired" speed distribution which is a function of density. (*5, 6*) From this, it follows that the average traffic speed would relax to a desired function of density. The actual traffic speed at any instant would be random; observed traffic speeds, however, would tend to cluster around the desired or "equilibrium" speed-density curve.

There is a vast and highly significant difference between a deterministic speed-density curve and a desired speed-density curve. In the former, all data points must lie exactly on the speed-density curve (within observational error). In the latter, only those traffic conditions that are "in equilibrium" lie on the curve. Considerable effort, therefore, may be required to extract the desired speed-density curve from data containing both transient and equilibrium observations.

Although the desired speed-density formulation is a substantial improvement on deterministic speed-density, only users of the Prigogine-Herman "Boltzmann-like" traffic theory seem to have adopted it.

[1] Italic numbers in parentheses identify references on page 96.

Moreover, the desired speed-density formulation has serious flaws which make it untenable. One of those flaws is the tendency to spontaneously "lock up"; other problems with the model will be discussed later.

Consider a highway section with moderate traffic flow where traffic density, speed, and volume vary from place to place and time to time. Consider, in particular, a short section of roadway which momentarily has higher density than the section just upstream of it. If the density is high, the speed will tend to be low. Thus, there is an upstream section of road pouring vehicles into a downstream section at high speed out of which latter section vehicles are tending to move at low speed. The density in the downstream section must therefore increase. If density increases, speed must tend to decrease even more. If the desired speed goes to zero at some "jam" density, traffic will lock up and no vehicles will ever escape. (Figure 1.)

While the density gradient above which traffic becomes unstable depends on the exact shape of the desired speed-density curve and other factors, any traffic model postulating that speed decreases as density increases has this inherent instability.

Payne's Traffic Hypothesis

Harold Payne developed a variant of the desired speed-density formulation which overcomes the spontaneous lock-up problem. (*7, 8*) Payne's formulation is far superior to either of its predecessors; it has not received the appreciation it deserves, perhaps because it was developed as a simulation program rather than as a traffic flow model. (*9*)

Payne's model adapts the desired speed-density formulation and adds a "look-ahead" term to the speed equation. The desired speed thus is the speed appropriate for the local density plus a term appropriate for the next downstream section. This takes advantage of the fact that if a section starts to lock up because of a local density increase, the density in the next downstream section of road will decrease. The desired speed in the first section is decreased because of its own large traffic density but increased because of the low density in the downstream section. This technique encourages standing queues to discharge when the obstruction to their flow is removed. (Such queues remain in place forever in the deterministic and desired speed-density formulations.)

A problem with both the "pure" desired speed-density and Payne's variant is that since the desired speed-density curve determines capacity and jam density, these items become merely "desired capacity" and "desired jam density." According to these formulations, it is possible for momentary flows to be substantially greater than capacity and densities to be hundreds of vehicles per lane-mile. (Impossibly high

Figure 1.—Vehicles slowing to join a traffic queue. Vehicles near camera are just below their free speed, whereas those in the middle distance have slowed to queue speed.

densities are a known flaw in the FREFLO simulation program based on Payne's formulation.) (*10*) Even though these values are transitory, such behavior nevertheless is unacceptable in any formulation meant to represent real traffic.

Solution

The approach used was to free the third equation-of-state from behaving like real traffic. Rather, the combination of all three equations-of-state must behave like real traffic. This represents a major departure from previous methods wherein the entire dependence of speed upon volume was attempted to be incorporated into a single statement independent of equations (1) and (2). In this research, various theoretically sound hypotheses were formulated and the resulting three equations-of-state were numerically integrated for a wide range of traffic conditions. Aberrations from realistic traffic performance were analyzed, and the hypotheses refined or discarded, until an acceptable traffic model was achieved.

The new formulation begins with the observation that any decrease in desired speed with increasing density is likely to lead to unrealistic traffic lock-ups. Logical analysis and some recent observations indicate that congestion does not reduce desired traffic speed—i.e., congestion may reduce actual traffic speed, but drivers continue to desire to travel at the facility's free-flow speed. (*11*) This can be expressed as follows:

$$\delta v/\delta t = v\, \delta v/\delta x = (F-v)/T, \qquad k<k_{jam} \qquad (3)$$

where

F = free-flow speed on the facility (mi/h),
T = relaxation time (hr), and
k_{jam} = jam density for traffic (veh/mi).

The term on the left of the equals sign is traffic acceleration as experienced in the frame of vehicles moving with the traffic. (This is the standard expression for acceleration as experienced in a moving system. For a brief derivation, see reference (*6*) page 13.) F – v is the difference between the vehicles' desired and actual speeds. T is the quantity which determines how quickly traffic will accelerate/decelerate if its actual speed is less/greater than its desired speed. (Quantities used in this way are generally called "relaxation times.") Although it seems likely that T depends upon v, several plausible forms for T(v) were simulated; none gave dramatically different traffic performance from T = constant. For simplicity, T therefore is taken in this article as a constant independent of v.

The overall equation thus says: "The acceleration that traffic experiences (left side) is proportional to the difference between its desired speed and its actual speed divided by some relaxation time (right side)."

The principal novelty in this equation is its postulation that F is a constant. F may depend upon location along the roadway and traffic composition, but it does not depend upon traffic density.

Given appropriate boundary conditions, the three equations-of-state selected provide an exact solution for speed, volume, and density at any place x and at any time t. The three equations behave very much like real traffic with "waves" and lifelike recovery from transient conditions.

Capacity limit

In both the deterministic and desired speed-density traffic formulations, road capacity is determined by the shape of the speed-density curve. The new formulation so far lacks any concept of capacity; therefore, this must be added as a constraint upon the three equations:

$$v \le \text{capacity}/k. \qquad (4)$$

This inequality states that traffic reduces its speed so flow through a bottleneck will not exceed the bottleneck's capacity.

Jam density

The capacity-constrained compressible fluid defined by the four equations is an excellent representation of traffic at low densities. It does not, however, reflect the fact that traffic behaves like an incompressible fluid when its density approaches some maximum k_{jam}.

It is simple to incorporate a suitable density constraint:

$$k \le k_{jam} \qquad (5)$$

Here, k_{jam} is taken as a constant which may change from place to place (depending upon the number of lanes) but does not depend upon traffic speed. (It is straightforward to extend the treatment to k_{jam} dependent upon speed.)

There is more to an incompressible fluid than a simple restriction upon density; incompressible fluids flow with constant volume. That is, when traffic has compressed to jam density and the vehicles at the head of the queue are passing through a bottleneck with volume Q, the traffic farther back in the queue is also traveling with the same volume Q. Traffic back in the queue is moving more slowly than traffic in the bottleneck because density is greater in the queue. (There are generally more lanes available for the queue than in the bottleneck.) (Figure 2.)

This constant-flow property is not incorporated in any of the above relationships. Equation (3), in fact, explicitly assumes that traffic is compressible. The correct replacement for equation (3) thus is:

$$\delta Q/\delta x = O, \qquad k = k_{jam} \qquad (6)$$

Relations (1) through (6) constitute a complete traffic formulation. Relations (1), (2), (4), and (5) always apply; equation (3) or equation (6) applies depending upon the value of k.

Figure 2.—Incident. Vehicles form queue at jam density upstream of incident. Capacity has been reduced to zero. Downstream vehicles have accelerated out of the picture.

Experimental Results

Almost all traffic models give reasonable results at low densities. This is because vehicle conservation is the controlling principle at low densities, and all reasonable traffic formulations incorporate that principle. It is at high densities that most traffic formulations become unrealistic. For that reason, the new traffic model was demonstrated with a simulated accident causing a traffic queue to form. Traffic demand is uniform throughout the simulation but capacity is reduced at a single point for 0.3 hr and desired speed is reduced for 0.4 mi starting at the time of the accident and remaining for 0.2 hr after the accident clears.

Example: Total freeway length: 13 miles

Total time simulated: 1.00 hr
Volume: 3,000 veh/hr
Capacity: 4,000 veh/hr everywhere except where capacity = 2,000 veh/hr at 5.0 mi from 0.2 hr to 0.5 hr

Free-flow speed: 63 mi/h everywhere except where free-flow speed = 50 mi/h, 4.9 to 5.3 mi from 0.2 to 0.7 hr.

Figure 3 shows how traffic density, speed, and volume behave in this accident scenario according to the new representation. A traffic queue grows back from the accident site until the accident clears. The queue then begins to dissipate with volume equal to restored capacity but does not clear completely until about 0.33 hr after capacity returns to normal (about 0.13 hr after desired speed returns to normal). All traffic behavior appears realistic except that when the accident clears, the entire queue simultaneously starts to move. In real traffic, however, such "starting waves" propagate backward with finite speed from the head of the queue. The infinite propagation speed is a result of modeling traffic queues as incompressible. A finite propagation speed in queues could be obtained by incorporating a finite "reaction time" into the new model, but that would complicate the formulation: The infinite propagation speed is adequate for most purposes.

Figure 4 shows how traffic density, speed, and volume behave in the accident scenario according to the Payne desired speed-density formulation. No queue forms as the volume of 3,000 veh/hr continues to flow through the 2,000 veh/hr bottleneck almost unimpeded. In fact, the accident is imperceptible but for a moderate increase in traffic density downstream of the accident. (Note that the density perturbation occurs downstream, rather than upstream, of the accident.) This dramatically illustrates a characteristic of desired speed-density traffic formulations: Because traffic relaxes to the desired speed gradually, it also relaxes to the local capacity gradually. In this accident scenario, the oncoming traffic encounters a short (0.1 mi) section of road with a lower capacity and immediately begins to adjust its characteristics to obey that capacity. A small decrease in

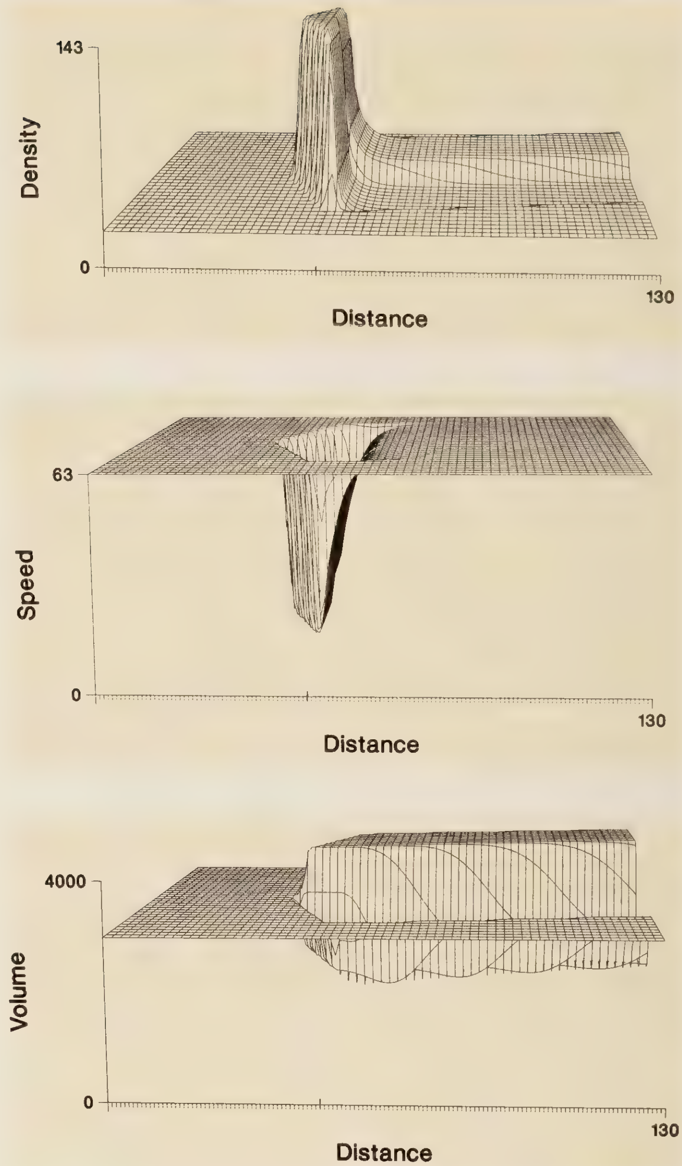

Figure 3.—Density, speed, and volume as represented by the new model when an incident reduces capacity from 4,000 to 2,000 veh/hr for 0.3 hr and free-flow speed from 63 to 50 mi/h for 0.5 hr. Demand volume is constant at 3,000 veh/hr. Traffic flows from left to right. Distance is in units of 0.1 mi (13 mi total). The distance tick mark at 50 (5.0 mi) is extended above the line to mark the incident location. Time runs from 0.00 to 1.00 hr front to back. Jam density of vehicles is 143 veh/lane-mi. Free speed is 63 mi/h.

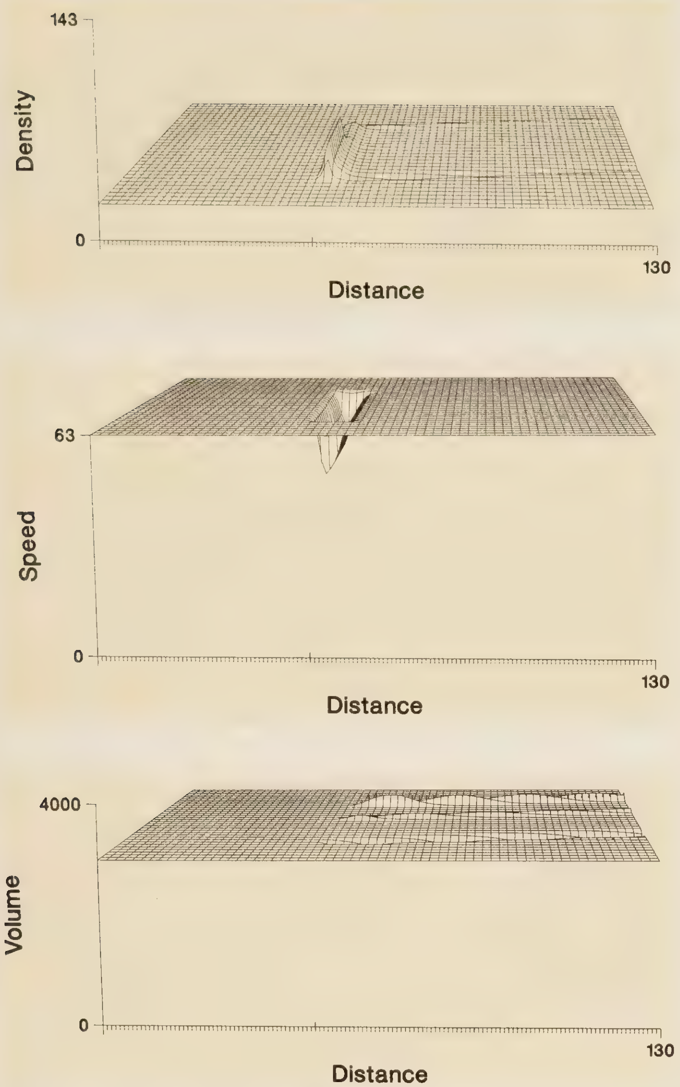

Figure 4.—Same conditions as in figure 3 except that traffic is represented by the Payne desired speed-density formulation.

volume occurs. In the next section, however, capacity rises back to 4,000 veh/hr and there is no further reason for traffic to reduce its flow. Almost no decrease in actual capacity is observed. If the reduced capacity section were extended, the traffic would eventually reduce its flow to match the correct capacity, but in this case the reduced capacity occurs on only a very short section. (Payne has informally referred to this as traffic being "sucked" through a bottleneck.) Desired speed-density formulations represent traffic very poorly in such scenarios.

The new traffic formulation has been tested on many other traffic scenarios, including on- and off-ramps, sections with reduced speed representing grades and poor road surfaces, and even traffic signals. Traffic representation appears quite realistic in every case.

Conclusions

The new traffic model discussed here reproduces traffic behavior more realistically than any previous macroscopic traffic formulation. Bottlenecks, queues, and free-flow conditions are well represented. "Starting waves" in queues are not, however, well represented.

The new traffic formulation reproduces the full range of traffic behavior from free flow to jam without any speed-density curve. The very concept of a speed-density curve is inconsistent with the new traffic formulation. While it is true that speeds are generally less than free-flow speeds at high densities, this is not because of any "speed-density curve," but is rather an effect of the changing volumes and densities whose interaction is given by equations (1) through (6). It is not possible to estimate a speed, for example, simply from the volume (or volume-to-capacity ratio). Instead, the speed at a point depends upon how long the volume has exceeded bottleneck capacity, how suddenly (and how often) the demand volume has changed and how long ago it changed. In short, the whole recent history of the traffic facility determines its present condition.

In its computer implementation, the new formulation serves as a macroscopic traffic simulation program. As such, it appears both simpler and more powerful than the FREFLO simulation program, the best previous macroscopic simulation program. The new formulation has the additional advantage of allowing larger integration steps.

The new formulation facilitates a new understanding of traffic. As such, its uses are innumerable and include its use in macroscopic traffic representations. In particular, the new model could be used in simulating freeways and medium to large-sized signalized networks. Such simulations will be faster and more accurate to implement than simulations based on previous models.

Control theory has not been applied to vehicular traffic control because traffic theory was so poorly defined. The new model is formulated in a way that makes it amenable to control theoretic analyses.

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(*10*) Ajay K. Rathi, Edward B. Lieberman, and Mark Yedlin, "Enhanced FREFLO Program: Simulation of Congested Environments," paper presented at the annual meeting of the *Transportation Research Board,* Washington, DC, January l987.

(*11*) Vanolin F. Hurdle and Dan Y. Solomon, "Service Functions for Urban Freeways—An Empirical Study," *Transportation Science,* Vol. 20, No. 3, August l986, pp. 153-163.

Paul Ross is a highway research engineer with the Office of Safety and Traffic Operations R&D, FHWA. He is involved with research on traffic capacity and levels of service. Before his assignment to "capacity," he was involved with research on traffic simulation, truck operations, and traffic signalization. Before joining FHWA, Dr. Ross was Assistant Professor of Physics at North Dakota State University.

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

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

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

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

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

Feasibility of Integrating Urban Traffic Operations Techniques, Vol. I, Executive Summary, Report No. FHWA/RD–87/021, and Vol. II, Final Report, Report No. FHWA/RD–87/022

by Traffic Systems Division

Urban traffic management systems—such as, traffic signals, freeway surveillance and control, motorist information, transit location, and parking advisory systems—can have a significant impact on traffic congestion in an urban area. In large urban areas, several of these types of systems may be in place or planned, each with its own hardware, communications system, and operating and maintenance personnel. This study investigated the feasibility of integrating the hardware or functional components of two or more traffic management systems in an urban area. Case studies were conducted in Cincinnati (OH), Rochester (NY), and Los Angeles (CA). The study concluded that system integration could be performed in many cases with significant cost savings and reduction in traffic congestion.

These reports may be purchased from the NTIS: (Vol. I, PB No. 87–193785, Price code: A02; and Vol. II, PB No. 87–193819, Price code: A01).

Promotional Issues Related to Off-Site Accident Investigation, Report No. FHWA/RD-87/036

by Traffic Safety Research Division

This report contains information of interest to highway administrators and transportation engineers relative to reducing congestion and related accidents by providing and encouraging the use of accident investigation locations off of and out of view of the freeway on which an accident has occurred. A number of vital legal, insurance, location, and administrative issues must be resolved for drivers—on their own to or aided by police—to promptly remove vehicles from the freeway to continue an accident investigation. This report includes a review of past research related to accident investigation sites (AIS) and results of interviews with concerned public and private agency officials. This material will be of assistance to officials interested in promoting a system of AIS adjacent to freeways within their jurisdictions.

Limited copies of this report are available from the RD&T Report Center. The report also may be purchased from the NTIS (PB No. 87–165320/AS, Price code: A03).

Seismic Design of Highway Bridge Foundations: Executive Summary, Report No. FHWA/RD-86/101; Design Procedures and Guidelines, Report No. FHWA/RD-86/102; and Example Problems and Sensitivity Studies, Final Report, Report No. FHWA/RD-86/103

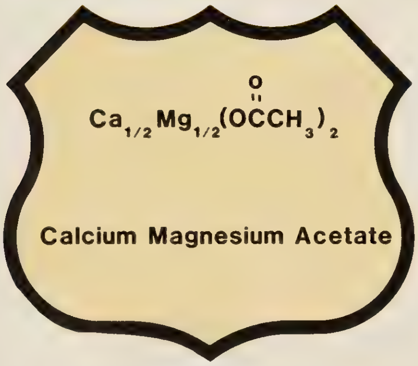
by Structures Division

This report provides guidelines for the seismic design of bridge foundations using design charts and computers, and is intended to supplement the *Seismic Design Guidelines for Highway Bridges* (published as a guide design specification by the American Association of State Highway and Transportation Officials in 1983). Design procedures are presented for footings, piles, drilled shafts, and abutments; and sample problem solutions, including some hand calculations, are contained in the appendixes. Comments on site investigation procedures and in situ and laboratory testing are provided in relation to determination of site soil parameters for analyses. Additional

comments are provided on earthquake-induced liquefaction and slope stability as they affect bridge foundation design.

Limited copies of these reports are available from the RD&T Report Center.

CMA Manufacture (II): Improved Bacterial Strain for Acetate Production, Final Report, Report No. FHWA/RD–86/117, and Executive Summary, Report No. FHWA/RD–86/181



by Materials Division

This report describes the study of three homoacetogenic anaerobic thermophilic bacteria for use in an industrial fermentative process to manufacture calcium magnesium acetate (CMA) from hydrolyzed corn starch and dolime. Batch fermentation, continuous fermentation with and without cell-recycling, and continuous fermentation with a new type of rotating fermenter were tested. Further research should lead to the development of an economic fermentation process for CMA.

The final report may be purchased from the NTIS (PB No. 87–191714/AS, Price code: A07). Limited copies of the Executive Summary are available from the RD&T Report Center.

Prefabricated Vertical Drains: Engineering Guidelines, Vol. I, Report No. FHWA/RD–86/168; Summary of Research Effort, Vol. II, Report No. FHWA/RD–86/169; and Laboratory Data Report, Vol. III, Report No. FHWA/RD–86/170

by Materials Division

These reports summarize the research and laboratory testing performed to develop procedures and guidelines applicable to the design and installation of prefabricated vertical drains to accelerate the consolidation of soils. The reports represent an interpretation of the state of the art as of August 1986. Volume I is intended to provide assistance to engineers in determining the applicability of PV drains to a given project and in the design of PV drain systems. Volume II presents a concise summary of the research effort and contains a statement of the research tasks and detailed description of each task. Volume III includes a statement of objectives, typical test results, a summary of procedures, and the actual laboratory test data. The information is intended for use by civil engineers familiar with the fundamentals of soil mechanics and the principles of precompression.

These reports may be purchased from the NTIS: (Vol. I, PB No. 87–154993/AS, Price code: A06; Vol. II, PB No. 87–192761/AS, Price code: A01; and Vol. III, PB No. 87–192779/AS, Price code: A01).

Geocomposite Drains, Vol. I: Engineering Assessment and Preliminary Guidelines, Report No. FHWA/RD–86/171; Vol. II: Laboratory Data Report, FHWA/RD–86/172

by Materials Division

These reports present summaries of laboratory testing and other relevant information available on geocomposite drain products, current research in testing of their critical properties, and design considerations including specifications. Volume I provides the engineer with a summary of currently available technical information and comments on the design and use of geocomposite drains. Volume II provides detailed laboratory test data on the measurement of critical properties of geocomposite drains.

These reports may be purchased from the NTIS: (Vol. I, PB No. 87–155008/AS, Price code: A05; Vol. II, PB No. 87–190682/AS, Price code: A01).

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

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

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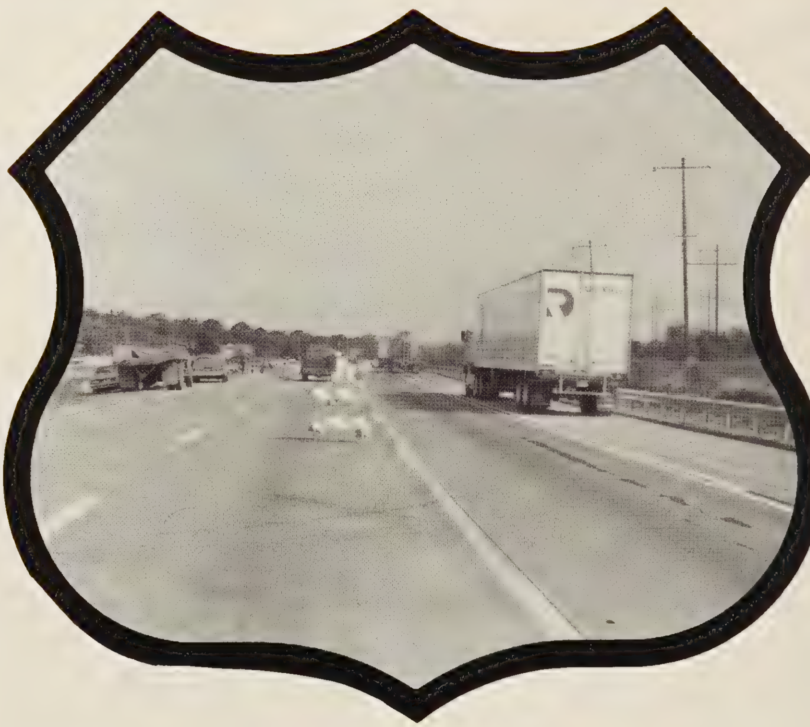


Speed Control Through Work Zones: Techniques Evaluation and Implementation Guidelines, Report No. FHWA–IP–87–4

by Office of Implementation

This report presents the implementation and evaluation of four techniques for improving the effectiveness of speed zoning in construction areas on multilane freeways: (1) The flagging procedure of the *Manual on Uniform Traffic Control Devices* (MUTCD); (2) the use of the MUTCD flagging procedures as well as motioning motorists to slow and pointing at a nearby speed limit sign with the free hand of the flagger; (3) a marked police car with cruiser lights and radar active; (4) a uniformed police officer standing to control traffic. A users guide on speed control in work zones is provided in Appendix B of the report.

This report may be purchased from the NTIS (PB No. 87–188041/AS, Price code: A04).

Computer Programs for Safety Analysis, Report No. FHWA–TS–87–211

by Office of Implementation

This report summarizes the implementation and enhancement process for the two computer programs—Highway Safety Analysis and Monitoring (HISAM) and Highway Safety Evaluation (HISAFE)— previously developed under this contract. The experience of the test city (Charlotte, NC) in implementing and operating the software is documented along with the advantages and limitations of each program. HISAM is used to maintain accident records, street inventory data, and traffic volume data; it can be used to identify high accident locations. HISAFE is used to perform safety evaluations of completed safety projects and conduct economic evaluations of the effects of the safety improvement. There are three related research reports: FHWA/RD–87/072, gives general background on the computer programs; FHWA/RD–87/073, documents and describes how to run HISAM; FHWA/RD–87/074, documents and describes how to run HISAFE.

These reports may be purchased from the NTIS:
(FHWA–TS–87–211, PB No. 87–183331, Price code: A03; FHWA/RD–87/072, PB No. 87–212825/AS, Price code: A04; FHWA/RD–87/073, PB No. 87–212833/AS, Price code: A03; FHWA/RD–87/074, PB No. 87–212841/AS, Price code: A07).

The computer software (available on IBM-compatible 5 1/4-in diskettes) and users manuals can be purchased from the Center for Microcomputers in Transportation (McTrans Center), 512 Weil Hall, University of Florida, Gainesville, Florida 32611 (telephone number 904–392–0378).

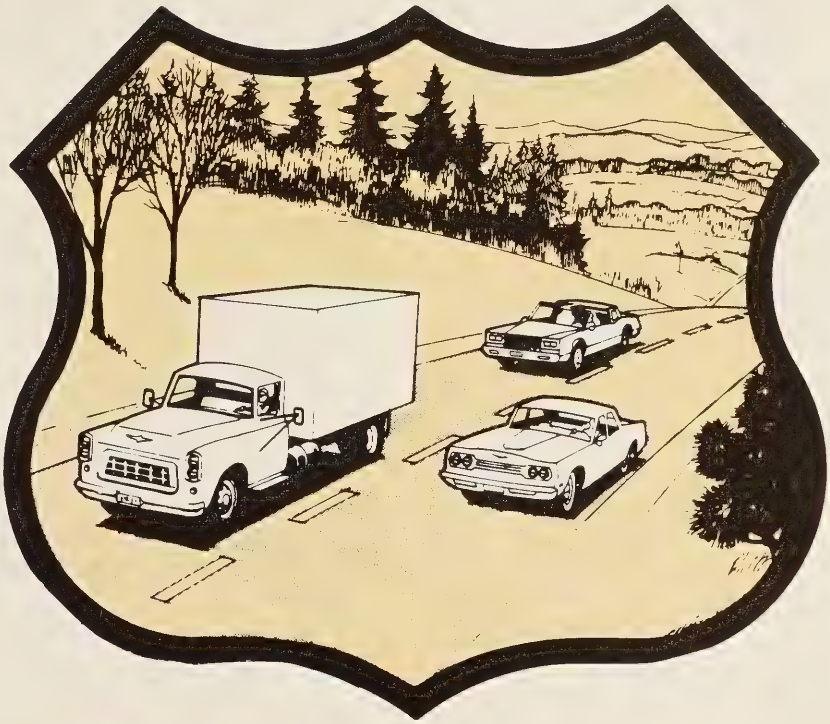
Limited copies of the report are available from the FHWA's Office of Traffic Operations (400 Seventh Street, HTO-21, Washington, DC 20590).

Review of State Comprehensive Computerized Highway Information System, Report No. FHWA–TS–87–220

by Office of Implementation

This report presents an overview of where the computerized highway information system is now, and its status as a planning and programming tool for State highway agencies. A computerized highway information system is simply a computer-linked system which can be used by many divisions of a transportation agency to obtain information to meet data reporting, analysis, or other informational needs. The description of the highway information system includes current use and status, applications, organization and system development, benefits, and problems.

This report may be purchased from the NTIS (PB No. 87–188033/AS, Price code: A03).



Pavement Marking Test and Evaluation Procedures, Report No. FHWA–TS–87–214

by Office of Implementation

This report includes data from an FHWA pavement marking study. One hundred forty-two liquid marking material systems were applied on both bituminous concrete and portland cement concrete surfaces during July 1986. Laboratory analysis of the paint systems were conducted over the next 6 months. Monthly field evaluations were made of the applied systems through February 1987.

Selection of Cost-Effective Countermeasures for Utility Pole Accidents, Users Manual, Report No. FHWA–IP–86–9

by Office of Implementation

This is a users manual to aid in the selection of cost-effective countermeasures for utility pole accidents. A manual cost-effectiveness procedure was developed which uses graphs, charts, and tables to allow for comparing accident benefits and project costs for such projects as burying utility lines, pole relocation, breakaway poles, and reducing pole density. The users manual also contains a discussion of user input variables and methods for establishing project priorities. A related report on this subject is "Utility Pole Accident Countermeasures Evaluation Program and Input Processor: Users Manual" (Report No. FHWA–IP–86–14).

This report may be purchased from the NTIS (PB No. 87–219382/AS, Price code: A09).

Optimization and Test of a Cavitation Water Jet, Report No. FHWA–TS–86–212

by Office of Implementation

This report describes an implementation project which had the objective to optimize the cavitation water jet and conduct field evaluations of the technology. Tests were performed in both the laboratory and the field. The report includes conclusions and recommendations resulting from the 25 tests performed on 12 concrete test samples in the laboratory and a series of tests performed at three test sites.

This report is available from the NTIS (PB No. 86–188034/AS, Price code: A03).

Utility Pole Accident Countermeasures Evaluation Program and Input Processor, Users Manual, Report No. FHWA–IP–86–14

by Office of Implementation

This is a users manual for the UPACE on IBM-PC and compatible microcomputers. The Utility Pole Accident Countermeasure Evaluation (UPACE) computer program is a tool to facilitate the cost-effectiveness analysis of utility pole accidents. The program undertakes analysis of roadway sections relative to utility pole accident problems and treatments, and provides the information needed for decision making. The UPACE program was originally developed for the mainframe computer. It has been converted to run under the MS DOS operating system on IBM-PC and compatible microcomputers. An input processor was developed to assist users in creating and modifying input data files. A related report on this subject is "Selection of Cost-Effective Countermeasures for Utility Pole Accidents, Users Manual" (Report No. FHWA–IP–86–9).

This report may be purchased from the NTIS (PB No. 87–219390/AS, Price code: A05).

The manual and the accompanying computer software may be purchased from the McTrans Center, University of Florida, 512 Weil Hall, Gainesville, Florida 32611. Telephone: (904) 392-0378

Proceedings: Fourth Annual Pedestrian Conference, Report No. FHWA–TS–84–218

by Office of Implementation

The Fourth Annual Pedestrian Conference, held September 20 through 23, 1983, at Boulder, Colorado, was divided into two sessions, Pedestrian Safety and Pedestrian Design. The safety portion presented the national perspective from the FHWA and the National Highway Traffic Safety Administration. Specific safety techniques were discussed in workshops with special presentations on seat belt usage and the pedestrian/alcohol programs. The design portion of the conference concentrated on pedestrian facility design throughout the world, including many innovative techniques for converting vehicle space to serve pedestrian needs.

This report may be purchased from the NTIS (PB No. 86–185048/AS, Price code: A16).

Rock Blasting, Report No. FHWA–TS–85–225

by Office of Implementation

This handbook is specifically designed as a guide to engineers and blasting practitioners working with highway applications. The handbook is a basic review of explosives and their characteristics, and provides criteria for selecting explosives. A simple step-by-step procedure is outlined to help the engineer review blasting submittals systematically. Several solved examples are presented in a manner to simplify the necessary calculations, with step-by-step procedures given where appropriate.

This report may be purchased from the NTIS (PB No. 87–164786/AS, Price code: A17).

New Research in Progress

The following new research studies reported by FHWA's Office of Research, Development, and Technology are sponsored in whole or in part with Federal highway funds. For further details on a particular study, please note the kind of study at the end of each description and contact the following: Staff and administrative contract research—*Public Roads* magazine; 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, DC 20418.

NCP Category A—Highway Safety

NCP Program A.1: Traffic Control for Safety

Title: Assessment of Current Speed Zoning Criteria. (NCP No. 3A1D0032)
Objective: Conduct speed studies on a national sample of streets and highways to determine the reasonableness of existing speed limits and the validity of current speed setting procedures. The speed studies will concentrate on non-55 mi/h roads. Factors influencing speeds and accident risk including traffic court decisions on speed limits will be investigated. The final product will be objective and quantifiable guidelines for establishing speed limits that could be generally applied by traffic engineers and public officials.
Performing Organization: Analysis Group, Inc., Charlotte, NC 28202
Expected Completion Date: October 1988
Estimated Cost: $216,000 (FHWA Administrative Contract)

NCP Program A.3: Highway Safety Analysis

Title: National Safety Information System. (NCP No. 3A3B0522)
Objective: Develop a multi-state data base from State accident, roadway, traffic, and related files to be used in safety problem identification and analysis. In Phase I, identify required data, develop file structures and select States able and willing to supply these data. In Phase II, develop prototype multi-state merged system with data obtained from four selected States.
Performing Organization: University of North Carolina, Chapel Hill, NC 27514
Expected Completion Date: January 1990
Estimated Cost: $327,000 (FHWA Administrative Contract)

Title: Application of New Accident Analysis Methodologies. (NCP No. 3A3B0042)
Objective: Demonstrate usefulness of new statistical techniques—such as, Bayesian estimates, exposure concepts, etc. Provide instructions to State highway agency personnel for using these techniques for highway safety analysis. Analyze specific safety problems using these techniques.
Performing Organization: Texas A&M Research Foundation, College Station, TX 77843
Expected Completion Date: December 1989
Estimated Cost: $90,000 (FHWA Administrative Contract)

NCP Program A.4: Special Highway Users

Title: Development of Truck Characteristics for Use in Highway Design and Operation. (NCP No. 3A4A3072)
Objective: Identify highway design and operation standards that are sensitive to truck performance characteristics. Determine the adequacy of those standards for trucks. Develop and assess new standards for those situations where the correct standards do not adequately address the current or anticipated future truck population.
Performing Organization: Midwest Research Institute, Kansas City, MO 64110
Expected Completion Date: August 1989
Estimated Cost: $282,000 (FHWA Administrative Contract)

Title: Examination of Truck Accidents on Urban Freeways. (NCP No. 3A4A1042)
Objective: Determine the nature and extent of urban truck freeway accidents and their consequences relative to vehicle type and traffic and roadway characteristics.
Performing Organization: Goodell Grivas, Inc., Southfield, MI 48075
Expected Completion Date: February 1989
Estimated Cost: $98,000 (FHWA Administrative Contract)

NCP Program A.5: Design

Title: Development of Additional FOIL Capabilities. (NCP No. 3A5F3052)
Objective: Upgrade Federal Outdoor Impact Laboratory (FOIL) at the FHWA's Turner–Fairbank Highway Research Center to provide additional testing capabilities: extend runout area to provide sufficient space for study of post impact behavior of vehicles; upgrade pendulum to model the 1,800-lb vehicle, the small car size of the 1980's; develop capability to collect overhead camera data which can be used to obtain vehicle crush and side intrusion data; and determine feasibility of testing 3,400-lb and 5,400-lb weight vehicles at the FOIL.
Performing Organization: Analysis Group, Inc., Washington, DC 20006
Expected Completion Date: February 1989
Estimated Cost: $209,000 (FHWA Administrative Contract)

NCP Category B—Traffic Operations

NCP Program B.1: Traffic Management Systems

Title: Development of WADS Image Recognition Algorithms. (NCP No. 3B1B0012)
Objective: Develop a set of improved image processing algorithms necessary for the successful operation of a TV-based wide area detection system (WADS). Develop algorithms for traffic data collection, traffic signal control, and freeway incident detection. Evaluate image recognition algorithms for these applications using videotaped data of traffic patterns and vehicles under different environmental conditions.
Performing Organization: University of Minnesota, Minneapolis, MN 55455
Expected Completion Date: May 1989
Estimated Cost: $265,000 (FHWA Administrative Contract)

NCP Category C—Pavements

NCP Program C.2: Evaluation of Flexible Pavements

Title: Asphalt Mix Design and Performance. (NCP No. 4C2B1112)
Objective: This research study will collect in situ data from asphalt concrete pavements that are performing well and pavements that are showing signs of distress. Information will include density, asphalt content, gradation, and characterization of the aggregate component of the mixture. This information will identify the relationships between age, climate, and asphalt mixture properties; and the relationship between traffic and density; and identification of acceptable asphalt contents for satisfactory mixture performance. A mix design procedure will be adopted that can be used to check mixtures at the mix plant and during construction.
Performing Organization: Purdue University, Lafayette, IN 47907
Funding Agency: Indiana Department of Transportation
Expected Completion Date: February 1990
Estimated Cost: $147,000 (HP&R)

Title: Inplace Properties of Base Course and Subgrades with Nondestructive Testing. (NCP No. 4C2B1102)
Objective: Provide the base and subgrade material properties, based on nondestructive testing measurements, for use in the new flexible pavement design system. Study all of Texas' major base materials. Provide procedures to collect and use subgrade material properties.
Performing Organization: Texas Transportation Institute, College Station, TX 77843
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1990
Estimated Cost: $349,000 (HP&R)

NCP Program C.3: Field and Laboratory Testing

Title: Development of Dynamic Analysis Techniques for Falling Weight Deflectometer Data. (NCP No. 4C3A1362)
Objective: Develop advanced techniques for processing falling weight deflectometer data and monitoring pavement response under dynamic loading. Develop computer models of pavement response to predict dynamic displacements, strains, and stresses based on linear elastic and nonlinear material response. Perform pavement monitoring by installing accelerometers or geophones on the pavement surface and lower layers. Record the time history of pavement deflection.
Performing Organization: Center for Transportation Research, Austin, TX 77812
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1991
Estimated Cost: $542,000 (HP&R)

NCP Category D—Structures

NCP Program D.1: Design

Title: Wall Thickness Criteria for Hollow Piers and Pylons. (NCP No. 4D1A4232)
Objective: Examine the effects of wall thickness on the strength and stiffness of both prestressed and nonprestressed hollow piers and pylons. Develop an analytical model to investigate the local stability, stiffness, and strength of hollow piers and pylons. Develop design recommendations for consideration by AASHTO.
Performing Organization: University of Texas, Austin, TX 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1990
Estimated Cost: $240,000 (HP&R)

Title: Bond and Anchorage of Epoxy-Coated Reinforcement. (NCP No. 4D1A3162)
Objective: Determine the effect of epoxy coating on the bond and anchorage of reinforcing bars. Develop an analytical model for the fundamental bond characteristics of coated rebars. Determine the effects of coated transverse reinforcement on splices and develop lengths of coated bars. Determine anchorage requirements of coated hooked bars. Develop design specifications.
Performing Organization: University of Texas, Austin, TX 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1990
Estimated Cost: $175,000 (HP&R)

Title: Seismic Response of Tieback Walls. (NCP No. 4D1A22142)
Objective: Model and analyze the interaction between soil and tieback walls under seismic loading, using the finite element computer program Flush. Design an instrumentation package for an existing wall and investigate the wall with Flush. Evaluate existing designs, identify conditions for which they may not be safe, and recommend changes in the Washington State Department of Transportation design procedures.
Performing Organization: University of Washington, Olympia, Washington 98501
Funding Agency: Washington State Department of Transportation
Expected Completion Date: June 1989
Estimated Cost: $85,000 (HP&R)

NCP Program D.2: Management

Title: Structural Modifications for Enhanced Aerodynamic Performance of Long-Span Bridges. (NCP No. 3D2C2022)
Objective: Synthesize and refine criteria for the retrofitting of wind-sensitive structures and structural members. Determine the types of wind response problems that have been typically exhibited by bridges; identify techniques that have been employed to improve performance; and identify typical details that have been used successfully to reduce adverse aerodynamic response. Develop guidelines for effective retrofit of structures which exhibit undesirable wind-induced vibrations.
Performing Organization: Applied Research Associates, Raleigh, NC 27615
Expected Completion Date: July 1989
Estimated Cost: $150,000 (FHWA Administrative Contract)

Title: Estimation of Bridge Design Life. (NCP No. 4D2D1072)
Objective: Develop a bridge design life estimation procedure sensitive to different materials and considering variable lives of bridge components. Develop bridge life estimates using deterioration rates, maintenance, loads, and inspection and inventory data.
Performing Organization: Pennsylvania Transportation Institute, University Park, PA 16802
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: August 1990
Estimated Cost: $150,000 (HP&R)

Title: Detecting Incipient Failures in Bridges. (NCP No. 4D2B1082)
Objective: Develop a bridge monitoring system to be used to detect incipient failures in bridges. The system will be based on the use of diagnostic dynamic testing and a spectrum analyzer and an expert system computer program.
Performing Organization: University of Texas, Austin, TX 78712
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: August 1990
Estimated Cost: $387,000 (HP&R)

NCP Program D.4: Corrosion Protection

Title: Salt Penetration and Corrosion in Prestressed Concrete Members. (NCP No. 3D4C0262)
Objective: Investigate the magnitude of the corrosion problem caused by the penetration of chloride ions from deicer application or exposure to a marine environment into substructure and superstructure prestressed concrete bridge components. Survey six prestressed bridges and based on the analysis of the collected field data, suggest a viable method for assessing the conditions of prestressing steel while rehabilitation with a suitable protective system is still feasible.
Performing Organization: Construction Technology Laboratory, Skokie, IL 60077
Expected Completion Date: September 1988
Estimated Cost: $101,000 (FHWA Administrative Contract)

NCP Program D.9: Technology Transfer for Structures

Title: Work Zone Traffic Control Delineation for Channelization. (NCP No. 3D9A0234)
Objective: Develop guidelines for the selection and application of drums, barricades, panels, cones, and tubes for varying work zone conditions for lane closures and median crossovers.
Performing Organization: Center for Applied Research, Great Falls, VA 22066
Expected Completion Date: January 1989
Estimated Cost: $245,000 (FHWA Administrative Contract)

NCP Category E—Materials and Operations

NCP Program E.1: Asphalt and Asphalt Mixtures

Title: Improved Asphalt Concrete Pavement Mixture Design Development and Verification. (NCP No. 3E1D2092)
Objective: Develop a method for designing and analyzing asphalt paving mixtures using a more theoretically sound basis for materials selection and a more realistic integration of asphalt mixture properties and structural pavement design and predictive performance algorithms. Tests include the indirect tensile creep and fatigue. The design procedure will be verified through an extensive field program.
Performing Organization: Texas Transportation Institute, College Station, TX 77843
Expected Completion Date: September 1989
Estimated Cost: $250,000 (HP&R)

NCP Program E.5: Maintenance Effectiveness

Title: Analysis of Pollution Controls for Bridge Painting. (NCP No. 4E5D1074)
Objective: Evaluate the costs and effectiveness of currently practiced and alternative technology for removal of bridge paint in preparation for repainting. Evaluate impacts to air and water quality and the quality of the surface preparation. Evaluate the Pennsylvania Department of Transportation's "Guidelines for Environmental Pollution Controls for Bridge Painting" to determine the effectiveness for controlling pollution and the impact to project costs. Examine options and develop recommendations for management of spent abrasives and paint generated during surface preparation.
Performing Organization: Stephen G. Pinney and Associates, Port St. Lucie, FL 33452
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: December 1988
Estimated Cost: $236,000 (HP&R)

☆U.S. Government Printing Office: 1987—202-880/80001

Title: Development of a Rational Approach to the Evaluation of Pavement Joint and Crack Sealing Material. (NCP No. 4E5C3642)
Objective: Develop guidelines for sealing cracks and joints in portland cement concrete and asphaltic concrete pavements to predict potential of materials for use as a crack or joint sealant; to select specific materials based on conditions of use; to specify installation techniques based on the materials involved and the conditions of use; and to evaluate the performance of the sealant in place.
Performing Organization: University of Cincinnati, Cincinnati, OH 45221
Funding Agency: Ohio Department of Transportation
Expected Completion Date: July 1989
Estimated Cost: $405,000 (HP&R)

NCP Program E.8: Construction Control and Management

Title: Impact of Aggregate Gradation and Type on Asphalt Concrete Mixture Characteristics and Pavement Performance. (NCP No. 4E8B1062)
Objective: Determine the importance of aggregate characteristics on the behavior of asphalt mixtures and pavement performance and recommend specifications. Investigate voids in the mineral aggregate (VMA), filler content, humps in the gradation curve, variation from specifications during production, and other areas.
Performing Organization: Center for Transportation Research, Austin, TX 78744
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: August 1990
Estimated Cost: $188,000 (HP&R)

Title: Highway Construction Cost Estimating Workstation. (NCP No. 4E8C2133)
Objective: Develop microcomputer programs for automating the preparation of construction cost estimates.
Performing Organization: Info Tech, Inc., Gainesville, FL 32604
Funding Agency: Pennsylvania Department of Transportation
Expected Completion Date: September 1988
Estimated Cost: $127,000 (HP&R)

Title: Development of Performance-Related Specifications for Portland Cement Concrete Pavement Construction. (NCP No. 3E8B2022)
Objective: Develop suitable conceptual framework of relationship between materials and construction test results and pavement performance for portland cement concrete pavements. Locate existing data that confirms some or all of these relationships. Outline laboratory and field testing program needed to supplement existing data. Conduct laboratory and field testing necessary to develop relationship between one material and construction variable and performance.
Performing Organization: Austin Research Engineers, Inc., Austin, TX 78746
Expected Completion Date: June 1989
Estimated Cost: $232,000 (FHWA Administrative Contract)

ERRATA

On page 51 of the September 1987 issue of *Public Roads* magazine, the y axis of figure 2 should read "Recognition Distance (thousands of feet)." The y axis of figure 3 should read "Legibility Distance (feet)."

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**Federal Highway
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