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**COVER:** Research studies conducted nationwide on the relationship between safety and cross section and roadway alinement elements are discussed in the second article in this issue.

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## IN THIS ISSUE

### Articles



### Departments



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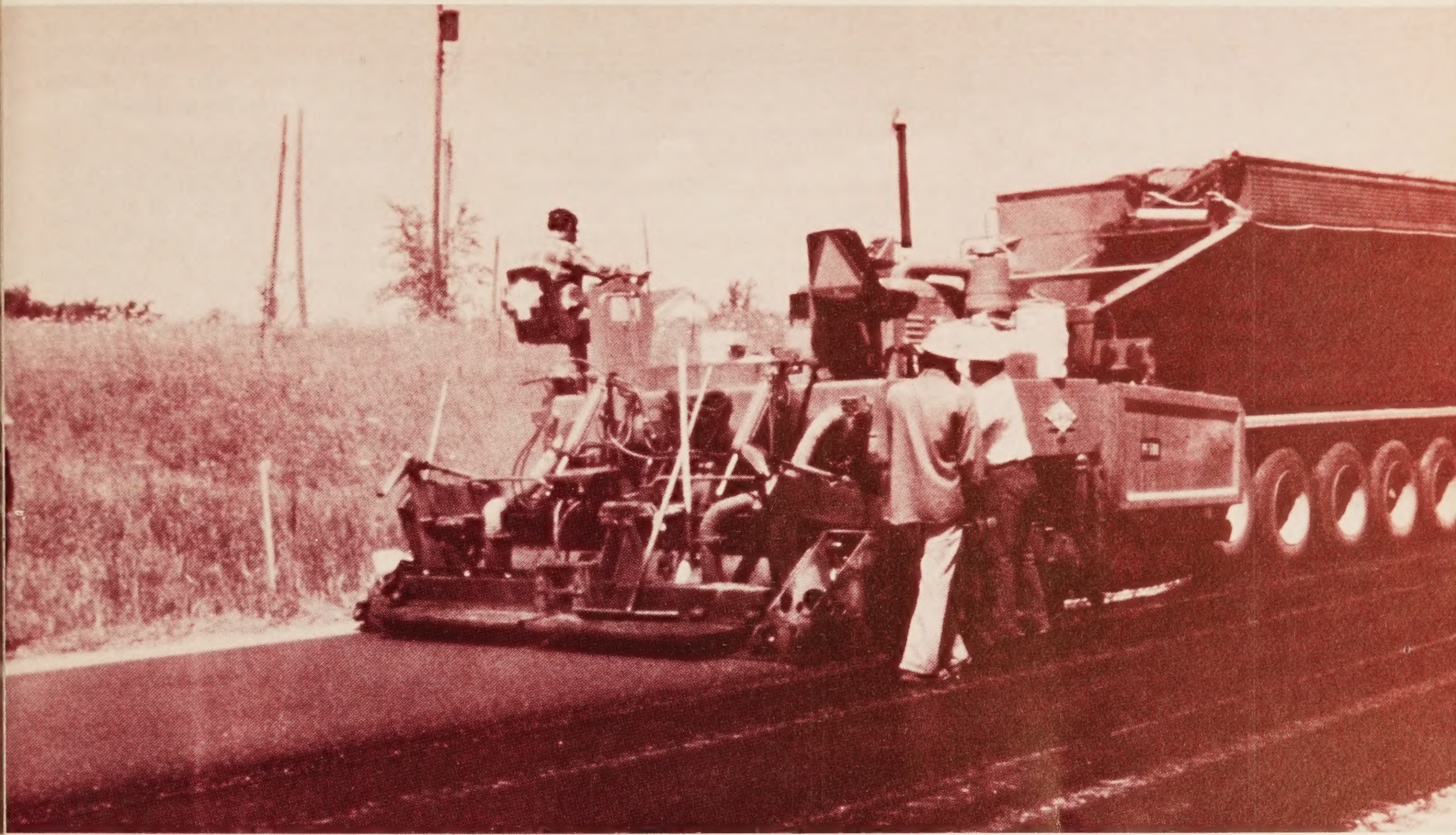

# Chemically Modified Sulfur Paving Binders

by
E. T. Harrigan and H. J. Lentz

**This article is based in part on the research paper, "Laboratory Evaluation of Sulphlex-233: Binder Properties and Mix Design," for which Dr. Harrigan and Mr. Lentz received the 1982 award in the annual outstanding paper competition held among the employees of the FHWA Offices of Research, Development, and Technology. The award was announced in the December 1982 issue of *Public Roads,* page 124.**

## Introduction

Chemically modified sulfur paving binders, formed by the reaction of elemental sulfur with one or more hydrocarbon or polysulfide modifiers at elevated temperatures, are materials designed to replace asphalt or portland cement in highway paving mixtures.

In general terms, chemically modified sulfur binders include the sulfur concretes principally developed to replace portland cement concrete in corrosive environments such as in ore smelting operations. A material of this kind is described in reference 1. Such materials may have application to highway construction, but specific highway trials of these materials have not been made.

The U.S. Department of Transportation, Federal Highway Administration (FHWA), is sponsoring research directed toward replacing asphalt in highway paving mixtures with chemically modified sulfur binders. Sulphlex[1] binders, characterized by high modifier loadings, are an important part of this research program. Typically, the starting components are 70 weight percent elemental sulfur reacted with 30 weight percent of two or more chemical modifiers. In contrast, the sulfur concretes may start with as much as 95 weight percent elemental sulfur in the binder. Another important characteristic is that Sulphlex paving mixtures contain 6 to 8 percent of binder by weight of

[1] The term "Sulphlex" is a registered trademark of the Southwest Research Institute, San Antonio, Tex. Sulphlex binders are covered by United States Patent 4,290,816.

the mixture; sulfur concretes typically contain 16 percent or more of binder by weight of the total mixture. Finally, the Sulphlex concretes are intended to be densified by compaction as are asphaltic concretes; high binder content sulfur concretes do not require such treatment.

Detailed information on the development, characteristics, and use of Sulphlex binders in paving mixtures is found in references 2 and 3. This article will discuss in detail the properties and use of only one material, Sulphlex binder number 233 (S–233). S–233 has been tested extensively in the laboratory and used in the construction of experimental pavement sections at seven locations in the United States.

## Composition and Physicochemical Properties

S–233 is the reaction product of elemental sulfur (70 weight percent) with a mixture of vinyl toluene (8 weight percent), dicyclopentadiene (12 weight percent), and dipentene (10 weight percent). The modifiers are used in commercial grades of purity and are available in bulk. The elemental sulfur is a very pure, run-of-the-mine grade. The combined modifiers are added to molten (149° C [300° F]) elemental sulfur in a reaction vessel over a period of 30 to 60 minutes. The reaction is moderately exothermic, and depending on the reaction temperature selected, acceptable product can be produced in as little as 2 hours.

As produced, S–233 is a pliable, asphalt-like material, and is the color of dark mahogany. When mixed with aggregate, it appears brown. Table 1 compares the physical properties of S–233 with those of an asphalt cement (AC–20 grade). Of particular importance in this table is the higher specific gravity of the S–233 and its lower flash and fire points. The S–233 is only partially soluble in trichloroethylene, and because of its high sulfur content, is readily soluble only in thiophene and carbon disulfide. S–233 and all other Sulphlex formulations examined to date harden at ambient temperature. This hardening differs dramatically in both degree and kind from the well-known oxidative hardening of asphalts. The S–233 hardens rapidly—a loss of 90 percent of the original penetration after 30 days aging is not uncommon. The hardening occurs because the S–233 contains an appreciable amount of unreacted elemental sulfur, originally present as dissolved liquid ($S_8$ units), which crystallizes with time. If a hardened S–233 sample is remelted, it regains its original penetration value.

S–233 also is susceptible to irreversible hardening caused by loss of volatiles during extended storage at high temperature. Table 2 presents results of the thin-film oven test[2] for S–233 compared with a typical AC–20 asphalt cement. The severe effect of this test on the S–233 is readily apparent. Of course, these effects are irreversible because they involve loss of volatile materials.

[2]The thin-film oven test, AASHTO T–179, is designed to simulate in the laboratory the effect of producing an asphalt-aggregate mixture in a typical hot-mix plant.

Thermogravimetric analysis of bulk S–233 indicates that upon heating, small weight loss begins at about 100° C (212° F), but substantial loss is delayed until 150° C (302° F) is reached. Above 200° C (392° F), the loss rate is accelerated and the material decomposes.

Gel permeation chromatography shows that, in general terms, the average molecular weight of S–233 (200 to 700 units) is considerably lower than that associated with asphalt. Considering the molecular weights of the original reactants, S–233 can be considered a polymeric material only in the widest sense of the term.

## Mixture Design and Characteristics

Research performed by FHWA to select a tentative aggregate mixture

**Table 1.—Comparison of binder properties**

| Property (test) | S–233 | AC–20 asphalt cement |
|---|---|---|
| Penetration at 25° C (AASHTO T–49) | 172 | 94 |
| Viscosity, absolute, 60° C, poise (AASHTO T–202) | 1,042 | 2,071 |
| Viscosity, kinematic, 135° C, cSt (AASHTO T–201) | 261 | 457 |
| Specific gravity at 25° C/25° C (AASHTO T–228) | 1.538 | 1.032 |
| Flash point, °F (AASHTO T–48 modified) | 355 | 505 |
| Fire point, °F (AASHTO T–48 modified) | 375 | 560 |

1 poise = 0.1 Pascal second
1 cSt = $10^{-6}$ metre$^2$/second
°C = (°F–32)/1.8

**Table 2.—Comparison of binder properties after thin-film oven test (AASHTO T–179)**

| Property (test) | S–233[1] | AC–20 asphalt cement[2] |
|---|---|---|
| Loss, weight percent | 2.64 | 0.73 |
| Penetration, 25° C (AASHTO T–49) | 51 | 50 |
| Percent penetration retained | 29.7 | 53.2 |
| Viscosity, absolute, 60° C, poise (AASHTO T–202) | 5,712 | 7,428 |
| Viscosity, kinematic, 135° C, cSt (AASHTO T–201) | 551 | 830 |

[1]Tested at 275° F.
[2]Tested at 325° F.

1 poise = 0.1 Pascal second
1 cSt = $10^{-6}$ metre$^2$/second
°C = (°F–32)/1.8

design method for S–233 is reported in reference 3. Because of the similarities in physical properties between S–233 and asphalt cement, the use of the Marshall design method and the Immersion-Compression test (to assess water damage susceptibility) to design S–233 mixtures was investigated. (Because Sulphlex binders and asphalt cements also are dissimilar in many physical properties, research is underway to develop a rational mixture design procedure for Sulphlex binders. Application of the Marshall design method to S–233 mixtures must be carefully qualified, because the field experience upon which this method is based for asphalt mixtures is lacking for Sulphlex.)

A traprock (diabase) aggregate used for road construction in northern Virginia and having a history of satisfactory service in asphaltic mixtures was chosen for this investigation. A dense gradation meeting the requirements of ASTM D–3515, Mix 6A, was selected and used in all binder-aggregate mixtures. Table 3 compares typical Marshall design method results for S–233 and an AC–20 asphalt cement with this aggregate and gradation. The S–233 mixture meets all the usual Marshall design criteria for asphalt mixtures and the method will yield "optimum" binder contents for S–233 using the standard procedure applied to asphalt mixtures. These results are contained in table 4. Based on this methodology, optimum S–233 mixtures are slightly greater than an equal volume replacement of asphalt cement. Because of the difference in specific gravities and loss upon heating between the two binders, this entails a 60 percent weight penalty for the S–233 with this particular aggregate.

Representative Immersion-Compression test data are presented in table 5 for both S–233 and the control AC–20 grade asphalt cement. It is evident that under the conditions of this test, the S–233 mixtures are susceptible to severe water damage. The use of tall oil improves the S–233 results dramatically, but the best S–233 behavior remains marginal compared with an untreated asphalt mixture with an approximately equal binder content. (An unfavorable reaction with the S–233 precluded the use of amine-type antistrip additives or hydrated lime.)

No precise correlation is available between the Immersion-Compression test results and the performance to be expected from pavements in the field. The test provides comparative guidance on water damage susceptibility. For this particular aggregate (table 5), the dry S–233 mixtures and the treated S–233 mixtures after immersion exceed the 2.1 MPa (300 psi) minimum compressive strength usually considered necessary for acceptable asphalt mixture performance in the field. Retained strengths of S–233 mixtures generally fall below the 70 percent criterion considered acceptable for asphalt mixtures.

Examination of S–233 mixtures with other aggregate types and gradations (as in the pavement construction discussed below) generally confirms the results presented here. S–233 mixtures can meet or exceed most Marshall design and compressive strength criteria at either equal volume or equal weight replacement of asphalt cement, but "optimum" mixtures are obtained at or near the equal volume replacement level. The S–233 mixtures appear susceptible to water damage (stripping), and the use of an antistrip treatment such as tall oil generally is recommended.

## Pavement Construction With S–233

The Sulphlex program is a long-range, high-risk research activity. FHWA judged it reasonable, however, to conduct limited construction operations with Sulphlex binders early in the development program. Such construction was intended to serve several purposes:

- Determine whether Sulphlex binders are compatible with the same equipment and techniques used in asphalt pavement construction.

**Table 3.—Typical Marshall design results: Sulphlex–233 and AC–20 asphalt cement (diabase aggregate)**

| Property | S–233 | AC–20 asphalt cement | Typical asphalt criteria |
|---|---|---|---|
| Weight percent | 9.6 | 6.1 | — |
| Volume percent | 16.8 | 15.6 | — |
| Percent air voids | 4.0 | 4.0 | 3–5 |
| Stability (lb) | 1,625 | 2,550 | 500+ |
| Flow (1/100 in) | 13 | 10 | 8–18 |
| Percent VMA | 18.6 | 17.3 | 16+ |
| Percent VMA filled | 78.5 | 76.9 | 75–85 |
| Unit weight ($lb/ft^3$) | 164.5 | 160.8 | — |

1 lb = 0.454 kg
1 in = 25.4 mm
1 $lb/ft^3$ = 16.018 $kg/m^3$

**Table 4.—"Optimum" Marshall binder contents (diabase aggregate)**

| | S–233 | AC–20 asphalt cement |
|---|---|---|
| Weight percent at maximum density | 11.4 | 6.9 |
| Weight percent at maximum stability | 10.0 | 6.3 |
| Weight percent at 4 percent air voids | 9.6 | 6.1 |
| "Optimum" content (weight percent) | 10.3 | 6.4 |
| "Optimum" content (volume percent) | 17.9 | 16.2 |
| Mixture properties at "optimum": | | |
| Percent air voids | 3.0 | 3.1 |
| Stability (lb) | 1,805 | 2,600 |
| Flow (1/100 in) | 15 | 12 |

1 lb = 0.454 kg
1 in = 25.4 mm

- Identify hidden problems with the field use of Sulphlex binders that would preclude their further development.
- Provide limited data on the durability and performance of Sulphlex binders in a variety of climates and traffic loadings.
- Provide some experience in the bulk production of Sulphlex binders.

An initial construction project took place on the grounds of the Southwest Research Institute in San Antonio, Tex., in December 1978. A 198 m (650 ft) long, 3.7 m (12 ft) wide pavement was built on a lightly traveled service road. Three Sulphlex binders, including S–233, were used in producing 10 Sulphlex pavement sections with a variety of thicknesses and binder contents. The construction took place on an asphalt-primed, compacted limestone rock base. The Sulphlex binders were produced at Southwest Research Institute in 3.4 Mg (7,500 lb) batches. Full details of the binder production and the construction can be found in reference 2.

A batch plant was used to produce the Sulphlex hot-mixes; a high quality, well graded limestone aggregate was used. The Sulphlex hot-mix was hauled 24 km (15 miles) in open trailers, placed with a conventional asphalt paver, and compacted with a vibratory, steel-wheeled roller.

The entire construction operation was accomplished without incident. The asphalt equipment and techniques worked as well with Sulphlex binders as with asphalt. After 3.5 years of service, with nearly 1,000 average daily traffic (ADT) volume (10 percent heavy trucks), distress of varying degrees is evident in several of the 10 test sections. The distress is related to several factors: poor subgrade drainage at the site, raveling of low binder content mixtures, lateral shifting of lifts because of the absence of tack coats between them, and placement of certain sections at extremely low mix temperatures (below 66° C [150° F]). For this particular project, none of the distress is uniquely related to the use of Sulphlex binders in lieu of asphalt cement in the pavement.

The success of the Southwest Research Institute construction and the apparent serviceability of the Sulphlex pavement sections spurred the organization of a program to construct Sulphlex pavement sections on public highways at various sites throughout the United States. Seven pavements were constructed during 1980 and 1981; details of the individual projects are contained in table 6.

The sites have diverse climates. The locations in Arizona, Florida, and Texas rarely experience temperatures below 0° C (32° F), and summer high temperatures range from 32° to 35° C (90° to 95° F) in Florida and Texas to well above 38° C (100° F) in Arizona. On the other hand, the winter low temperatures in North Dakota, Nebraska, Michigan, and Pennsylvania can fall to –23° to –29° C (–10° to –20° F) or lower; summer highs can reach 32° C (90° F) or above. Traffic volumes range from 3,000 to 6,000 ADT with the exceptions of Nebraska (250 ADT) and Florida (8,850 ADT per lane). The Florida and North Dakota sites are four-lane-divided facilities. The other five sites are two-lane facilities.

Sulphlex binder S–233 was chosen for use in the State construction projects. Approximately 180 Mg (200 tons) were produced at a chemical process plant in Odessa, Tex. Two production runs, made in August and October 1980, consisted of several 6.8 Mg (7.5 ton) batches. Because of a shortage of the preferred commercial grade of the modifier dipentene, a different commercial source was substituted.

**Table 5.—Immersion-Compression test data (AASHTO T-165, T-167): diabase aggregate**

| Volume percent binder | Weight percent binder | Percent air voids | Additive | Compressive strength, psi: Dry | Wet | Percent retained |
|---|---|---|---|---|---|---|
| S–233: | | | | | | |
| 11.0 | 6.1 | 12.4 | None | 744 | Fell apart | — |
| 11.0 | 6.1 | 11.2 | 1 percent tall oil | 708 | 301 | 42.5 |
| 16.0 | 9.1 | 6.4 | None | 657 | Fell apart | — |
| 16.0 | 9.1 | 5.0 | 1 percent tall oil | 681 | 478 | 70.2 |
| AC–20 asphalt cement: | | | | | | |
| 10.6 | 4.0 | 11.4 | None | 335 | 161 | 48.1 |
| 15.6 | 6.1 | 5.3 | None | 351 | 270 | 76.9 |

1 psi = 6.895 kPa

**Table 6.—Sulphlex construction projects, 1980–1981**

| State | Location | Approximate length | Binder requirement | Date | Pavement type[1] | Target binder content[2] |
|---|---|---|---|---|---|---|
| | | *Feet* | *Tons* | | | |
| Texas | Loop 1604, San Antonio | 7,467 | 45 | 8/26/80 | 1-in overlay | 7.0, 7.5, 8.0 |
| North Dakota | U.S. 2 and 52, Minot | 810 | 13 | 9/18/80 | 2-in wearing course | 6.0, 9.0 |
| Nebraska | N–66, Valparaiso | 920 | 22 | 9/19/80 | 2-in wearing course | 5.7 |
| Pennsylvania | TR405, Montgomery | 2,332 | 22 | 10/27/80 | 1.5-in wearing course | 10.0 |
| Florida | I–75, Gainesville | 646 | 21 | 11/20/80 | 5-in surface course overlay | 8.5 |
| Michigan | M–54, Grand Blanc | 740[3] | 22 | 7/30/81 | 2.5-in overlay | 7.5 |
| Arizona | U.S. 70, Safford | 675 | 13 | 11/12/81 | 2.5-in overlay | 8.6 |

[1]All nominal 12-ft wide lanes.
[2]Percent by weight of total mix.
[3]By 24 ft wide.

1 ft = 0.305 m
1 ton = 0.907 Mg
1 in = 25.4 mm

This substitution required a minor change in the relative proportions of the sulfur and modifiers used in the production, but the physical properties of the binder (designated S–233/A) were not altered significantly.

The mix designs for each construction were made jointly by FHWA and the State highway agency. As discussed above, the Marshall design method and the Immersion-Compression test for water damage susceptibility were used. The aggregate type and gradation used were identical in each case to that used in the asphaltic construction project of which the Sulphlex section forms a part. The S–233/A mix design usually required a binder content, which provides an equal volume of S–233/A binder as the optimum asphalt design. The immersion-compression testing indicated the desirability of treating the S–233/A mixes with a tall oil product to reduce their stripping potential. However, tall oil was added to the Sulphlex binder as an antistrip agent only in Florida and Arizona.

Batch-type plants were used for hot-mix production in Texas, Pennsylvania, Florida, and Michigan; drum mixers were used in the other States. The use of the S–233/A binder did not require the modification of any of the hot-mix plants. Production of the S–233/A hot-mix proceeded in the typical fashion for asphalt hot-mix. The S–233/A binder was used over the temperature range of 107° to 141° C (225° to 285° F) while the aggregates were heated to 141° to 160° C (285° to 320° F). The S–233/A hot-mix temperature fell in the range of 113° to 146° C (235° to 295° F).

In each State, the S–233/A hot-mix was hauled from plant to construction site in the kinds of trucks normally used by the contractor for asphalt hot-mix, including 45.4 Mg (50 ton) single units in Michigan and bottom-dump units in Arizona. The usual haul distance was 3.2 to 6.4 km (2 to 4 miles), but in Michigan and Pennsylvania the haul distances were 64 km (40 miles). Laydown (fig. 1) and compaction (fig. 2) were accomplished in all cases with conventional pavers and both steel-wheeled and pneumatic-tired

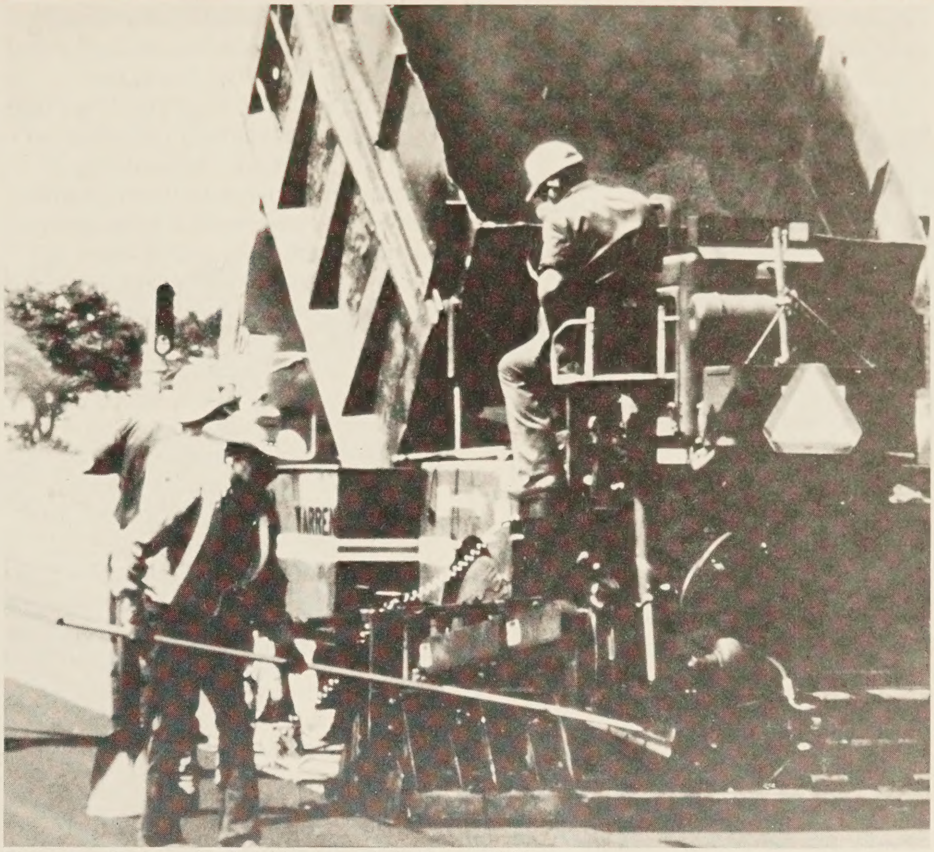

*Figure 1.—Paving with Sulphlex hot-mix in Texas, 1980.*

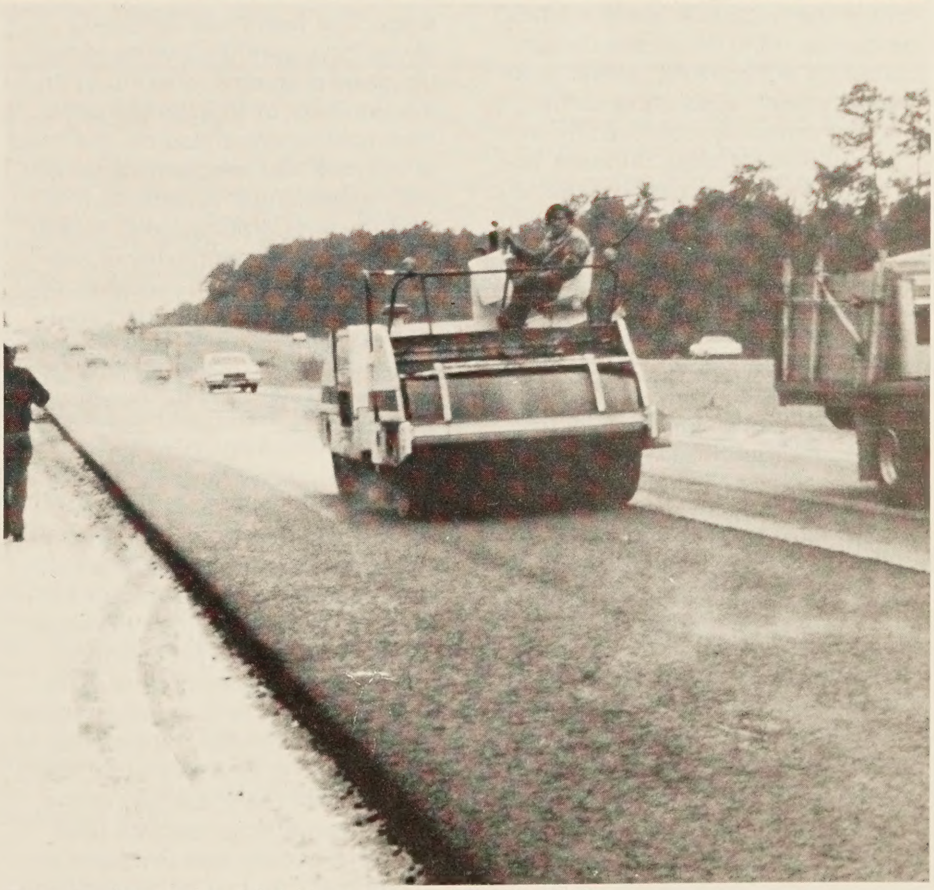

*Figure 2.—Compaction of Sulphlex pavement in Florida, 1980.*

rollers. Satisfactory compaction was attained using the same rolling pattern used for asphalt mixes at each project. The completed Sulphlex pavement in Texas is shown in figure 3.

Because all Sulphlex binders are composed predominately of sulfur, the emission levels of the toxic gases hydrogen sulfide ($H_2S$) and sulfur dioxide ($SO_2$) were monitored during the construction in each State. The most extensive and precise measurements were made in Michigan and Arizona—$H_2S$ levels at the construction site did not exceed 1.0 ppm and $SO_2$ levels did not exceed 0.25 ppm. Measurements at the hot-mix plant indicate that in normal operations with S-233/A binder, $H_2S$ levels can be maintained well below safety threshold limits in all usual working areas. Indeed, in all but a few areas, $H_2S$ levels below detectable limits should be realized.

Significant problems related to S-233/A hot-mix production were evident in North Dakota and Nebraska where drum mixer plants were used and actual binder content of the S-233/A hot-mix fell 50 percent or more below the design content. Consequently, in Nebraska the pavement failed under traffic, and the contractor removed more than 70 percent after 24 hours. In North Dakota the pavement raveled badly but has not shown significant thermal cracking, presumably because it was stabilized by fog-sealing with a liquid asphalt. The causes of these inadequate binder contents are unknown; speculation centers on an inability to adequately adjust the control functions of the particular drum mixer plants for the high specific gravity (about 1.5) of the S-233/A binder. In the Arizona construction, satisfactory binder content was achieved in drum mixer operations using the existing plant control functions.

There is a clear dichotomy between the performance of the S-233/A pavements in the southern climates and those in the northern climates. Where the weather is rarely or never below freezing, the S-233/A pavements showed no significant distress of any kind as of September 1982. On the other hand, the pavements in Michigan and Pennsylvania have developed reflective and thermal cracking. The situation is particularly severe in Pennsylvania where the pavement shows extensive map-cracking and is close to failure after two winters of service. This cracking behavior may be attributed to the stiff nature of the S-233/A mixtures compared with otherwise comparable asphalt mixtures. This stiffness is characterized both by the high resilient modulus of the S-233/A mixtures at low temperatures and by the high limiting stiffness temperatures calculated for S-233/A binder on the basis of its temperature susceptibility. (See reference 4 for a complete discussion of this topic.) Both parameters suggest that S-233/A mixtures will be susceptible to thermal cracking.

## Research in Progress

The technology for using chemically modified sulfur binder in pavement construction is at a rudimentary stage. The field trials described above have identified some of the unsolved problems inherent in the development of Sulphlex binders. Foremost among these are the need to improve low temperature behavior, reduce binder volatilization at working temperatures, and reduce the stripping susceptibility of Sulphlex mixtures. Such problems are potentially solvable yet their solutions require a long term, comprehensive research program, which in many respects must be considered speculative.

Another finding of the construction program was the batch-to-batch variability in the physicochemical properties of the S-233/A binder. The material was produced in 6.8 Mg (7.5 ton) batches from a recipe provided by FHWA. No specifications or quality control tests were available at the time, nor was there a method to monitor the progress of the reaction in situ. Moreover, the sensitivity of the properties of the finished product to reaction time and temperature was unknown so the viscosity and aging characteristics of different batches of S-233/A binder varied widely. (Blending of the batches reduced considerably the effects of batch-to-batch variation in the field.)

The current Sulphlex research program addresses the problems discussed above. Activities include the chemical characterization of Sulphlex binders as a class to assure quality control in their manufacture and use; materials characterization of Sulphlex-aggregate mixtures, including mix design techniques, static and dynamic testing, assessment of fatigue and creep properties of Sulphlex mixtures, and development of pavement designs to exploit specific Sulphlex properties; an environmental and safety assessment of Sulphlex binder use; and development of an efficient, economical process design for large-scale Sulphlex binder production.

Finally, work is underway on the development of a second generation of Sulphlex binders with markedly improved engineering and handling characteristics compared with the original materials such as S-233 and S-233/A.

Assuming reasonable progress in current research activities, future activities would include developing quality control testing procedures for Sulphlex binders and aggregate mixtures, assembling and operating a pilot plant to produce Sulphlex materials in significant tonnages, and evaluating several new materials and design procedures at large-scale pavement construction projects. Such construction, as the last step in the Sulphlex research program, would not take place before 1986 or 1987.

*Figure 3.—Sulphlex pavement, Loop 1604, San Antonio, Tex., September 1980.*

## Conclusions

Chemically modified sulfur binders, such as the Sulphlex binders discussed in this article, have the potential to replace asphalt in conventional bituminous paving mixtures. However, extensive laboratory research and large-scale field trials will be necessary before such binders can be promoted for routine use in pavement construction. Under the best of circumstances, routine use cannot be contemplated before 1990 and then only if asphalt supplies are inadequate to meet normal demands. In the interim, it is conceivable that Sulphlex or similar binders could be used on an emergency basis if asphalt supply were severely disrupted.

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(*3*) H. J. Lentz and E. T. Harrigan, "Laboratory Evaluation of Sulphlex–233: Binder Properties and Mix Design," Report No. FHWA/RD–80/146, *Federal Highway Administration,* Washington, D.C., January 1981.

(*4*) "Design Techniques to Minimize Low Temperature Asphalt Pavement Transverse Cracking," Research Report No. 81–1, *The Asphalt Institute,* December 1981.

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

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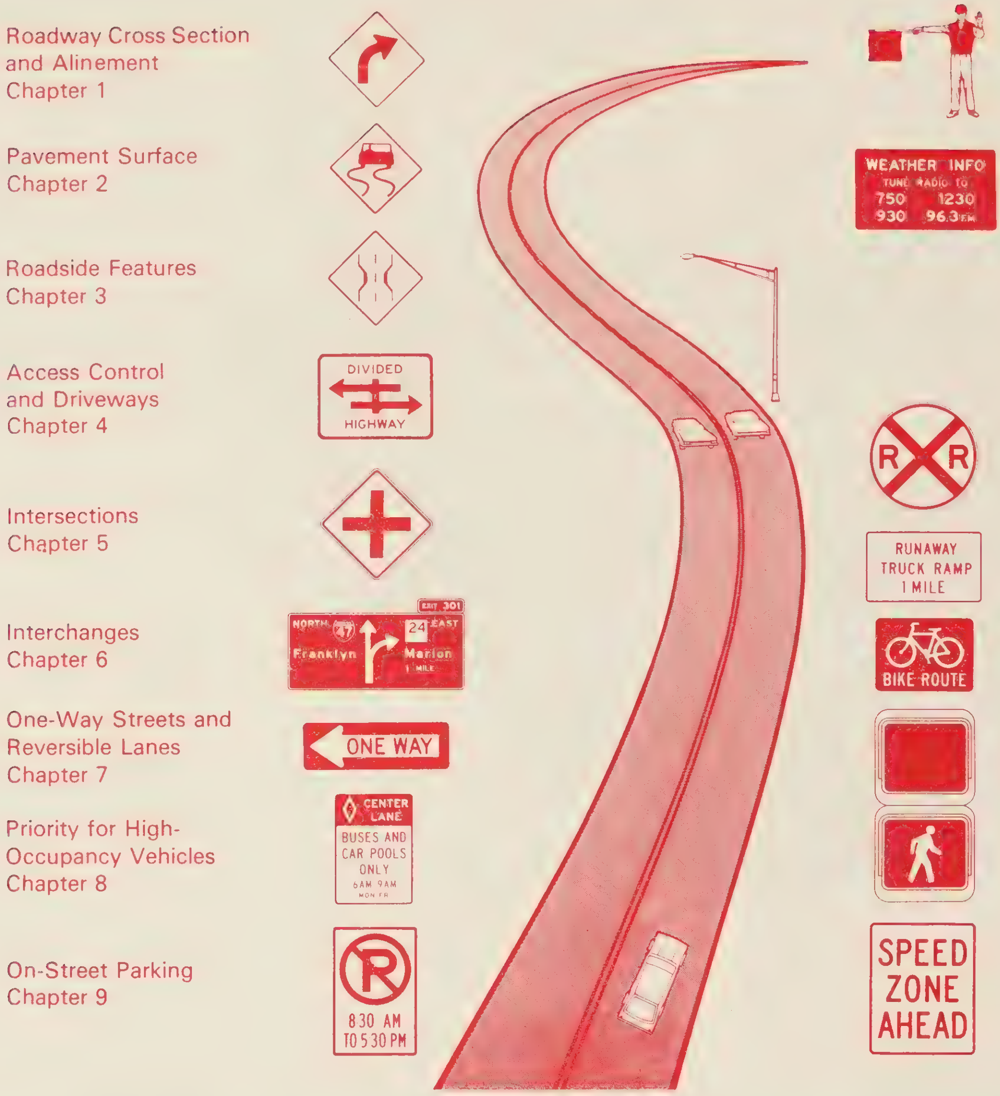


# Roadway Cross Section and Alinement

by

Howard H. Bissell, George B. Pilkington II, John M. Mason, and Donald L. Woods

This article is a condensation of the first chapter of "Synthesis of Safety Research Related to Traffic Control and Roadway Elements," a two-volume literature review published by the Federal Highway Administration (FHWA) (Report Nos. FHWA–TS–82–232 and –233).

Seventeen safety research subject areas are presented as individual chapters, authored by FHWA staff, Texas Transportation Institute staff, and other technical experts. Subject areas included in Volume 1 are roadway cross section and alinement, pavement surfaces, roadside features, access control and driveways, intersections, interchanges, one-way streets and reversible lanes, priority for high occupancy vehicles, and onstreet parking.

Volume 2 subject areas include construction and maintenance zones, adverse environmental operations, roadway lighting, railroad-highway grade crossings, commercial vehicles, bicycle ways, pedestrian ways, and speed zoning and control.

An overall subject index is included in both volumes of the synthesis.

The synthesis is available from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Vol. 1, Stock No. 050-001-00259-1 [$7] and Vol. 2, Stock No. 050-001-00260-4 [$6.50]).

## Introduction

The fundamentals of highway design include selecting physical dimensions that will appropriately serve the expected flow of traffic, considering ground topography along the designated route. The basic geometric dimensions involve the roadway cross section and the alinement. Roadway cross section includes pavement lane width, shoulder width, median width and shape if the highway is divided, cross slope of the pavement and slope of embankments, and kinds of draining ditches.

The roadway alinement along the selected route includes straight sections (tangents), horizontal and vertical curves, and roadway grades. Research has been conducted to evaluate the safety of these roadway elements by analyzing accident data on existing roadways.

The American Association of State Highway and Transportation Officials (AASHTO) has set geometric standards for various roadway types. (*1, 2*)[1] Some of the standards set by AASHTO are based on past research; others are from rational analyses and expert opinions. This article summarizes the relationships between safety, as measured by accidents, and the geometric cross section and roadway alinement elements.

When the roadway alinement or changes in the cross section are hazardous to the motorist, traffic control devices are installed to warn drivers of possible hazards or modify vehicle operation. Standards for these traffic control devices are set. (*3*) Research has been conducted on some of these devices to relate their effects on accident experience and vehicle operations.

## Cross Section

The cross section of a roadway can be represented by a slice made across the roadway, perpendicular to its alinement so that a profile is shown along the cutline (fig. 1).

[1]Italic numbers in parentheses identify references on pages 140–141.

The widths and slopes of component portions of the cross section affect vehicle operations and safety. Studies have been conducted on different component elements of the cross section as well as on combinations of the various elements.

Five recent studies have been conducted to determine the relationship of cross section features with accident data.

- Research was conducted in Louisiana to determine accident rates related to rural highway geometry. Accident data were collected for 246 sections of rural roadway where sections varied from 1.6 to 27.4 km (1 to 17 miles). Over 6,000 accident reports from 1962 to 1966 were reviewed. (*4*)
- A representative sample of 2 253 km (1,400 miles) from the 22 470 km (13,962 miles) of two-lane State highways in Ohio was studied in detail. Traffic accident records from 1969 and 1970 were used to identify the single-vehicle accident for the sample. The effects of roadway width and shoulder widths on accident rates were determined as well as studying the interaction of shoulder quality and the recovery area. (*5*)
- Data bases from the States of Maryland and Washington were combined to study the safety effect of various highway geometric features. The data base included 3 117 km (1,937 miles) of two-lane rural roads in Maryland and 3 235 km (2,010 miles) in Washington. The combined number of study sections was over 34,000 for the two States. (*6*)
- Accidents relating to lane and shoulder widths for two-lane rural roads in Kentucky were analyzed. Sampled were 15,994 1-mile sections and 16,760 accidents reported in 1976. Variables were controlled for classification and nonhomogeneous sections were eliminated. (*7*)
- In 1974 and 1975, the Arizona Department of Transportation improved shoulders on 178.9 km (111.8 miles) of two-lane highways. There were 98 accidents 1 year before; 1 year after the improvement, there were 92 accidents. The accident rate decreased from 1.12 accidents per million vehicle kilometres (A/MVkm) (1.81 accidents per million vehicle miles [A/MVM]) to 0.81 A/MVkm

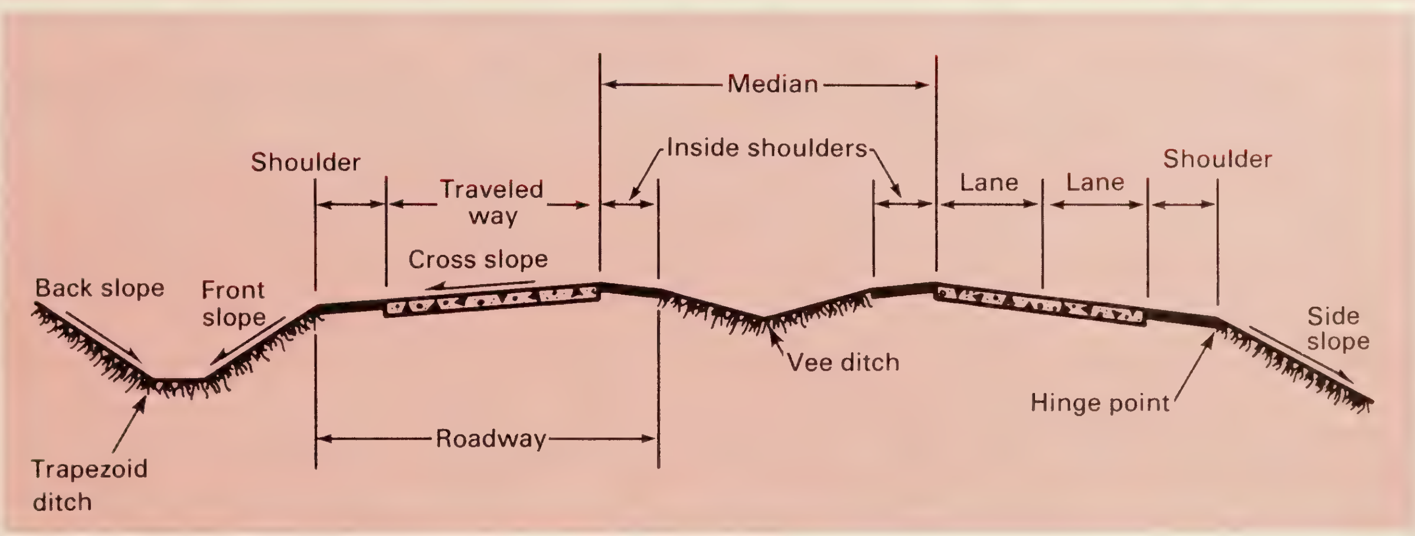


*Figure 1.—Typical cross section of a divided highway.*

(1.30 A/MVM). Run-off-the-road accident rates went from 0.5 to 0.36 A/MVkm (0.81 to 0.58 A/MVM). For 119 km (74 miles) of pavement widening projects done in 1974–1976, the accident rates fell from 1.17 to 0.71 A/MVkm (1.89 to 1.14 A/MVM) and the run-off-the-road accidents went from 0.58 to 0.30 A/MVkm (0.94 to 0.49 A/MVM). (*8*)

### Lane width

Research studies reported above have generally shown that accident rates decrease with an increase in the width of the traffic lane up to about 3.4 m (11 ft) on two-lane rural roads. (*4–8*)

The Alabama Highway Department studied the effects of widening lanes on two-lane rural highways. There were 17 sites where lanes were widened from 2.7 and 3.0 m (9 and 10 ft) lanes to 3.4 and 3.7 m (11 and 12 ft) lanes. There also were control sections and parallel study sections. The results showed a lane width increase reduced the injury-fatality accident rate significantly (22 percent) and caused a decrease in the total accident rate. (*9*)

Most high-volume roadway facilities were built with standard 3.7 m (12 ft) lanes. However, freeway sections were becoming congested during peak traffic periods. To increase the capacity through the bottleneck section, the pavements were restriped to add another lane by using narrower lanes and reducing shoulder widths. The improvement in traffic flow resulted in fewer accidents. (*10*)

### Shoulder width

Research on shoulder width related to traffic accident rates has had mixed results. Early research had indicated that accidents increase with increasing shoulder width. (*11–13*) More recent studies generally have shown that the accident rates have been reduced as shoulder widths increase to about 2.4 m (8 ft) on two-lane rural roads. (*5–7*)

An economic analysis of shoulder widening showed that the benefit-cost ratio is more than one for widening narrow shoulders on sections of two-lane rural roads having four or more run-off-the-road and head-on accidents per kilometre per year (six or more per mile per year). Shoulder widening would not be cost effective for low-volume roads (less than 1,000 vehicles per day) having low accident frequencies. No additional benefit is obtained on rural, two-lane roads by widening shoulders to more than 2.7 m (9 ft). Accident reductions for shoulder widening are shown in table 1. Higher priority should be given to shoulder widening on horizontal curves and winding sections than to shoulder widening on straight, level tangents. (*7, 12*)

### Shoulder surface treatment

In a 1974 study it was concluded that it was more cost effective for Ohio to improve (stabilize) two-lane rural road shoulders than to widen the pavement or clear the roadside. This was based on an analysis that unstabilized shoulders have 30 to 50 percent higher accident rates than stabilized shoulders, and the costs to stabilize shoulders are much less than pavement widening or roadside clearance. (*5*)

Based on a sample of 3,054 individual roadway sections (including two-, four-, and six-lane highways) in North Carolina, a significantly lower accident experience (about 20 percent) was observed for all kinds of highways with paved 0.9 to 1.2 m (3 to 4 ft) shoulders as compared with identical highways with unpaved shoulders. (*14*)

The Arizona Department of Transportation evaluated a 152 mm wide, 19 mm deep (6 in wide, ¾ in deep), 45° diagonal grooving of the right shoulder at 30 m (100 ft) intervals on a 16 km (10 mile) section of Interstate 8. Run-off-the-road accidents were reduced 61 percent. The cost of shoulder grooving the section was $2,000. It was estimated that the grooves saved 13 single-vehicle accidents in 3 years. Considering a 10-year life of the grooving, the benefit-cost ratio for the project was 108. A similar study was conducted to evaluate painted diagonal striping of the shoulder. The painted shoulder did not lower the accident rate. (*8*)

**Table 1.—Reduction in accident rates from shoulder widening on two-lane, rural roads (*7*)**

| Shoulder width: Before | Shoulder width: After | Reduction in run-off-the-road and opposite direction accidents |
|---|---|---|
| *Feet* | *Feet* | *Percent* |
| None | 1-3 | 6 |
| None | 4-6 | 15 |
| None | 6-9 | 21 |
| 1-3 | 4-6 | 10 |
| 1-3 | 7-9 | 16 |
| 4-6 | 7-9 | 8 |

1 ft=0.3 m

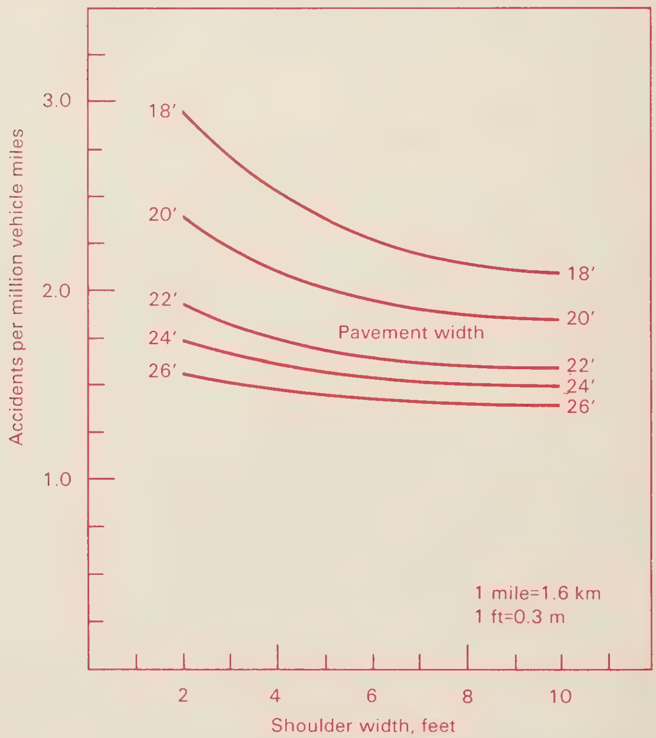


*Figure 2.—Accident rate for variable pavement and shoulder widths on two-lane rural highways. (15)*

## Roadway width

Shoulder width and pavement widths are related to expected accident rates in figure 2. (*15*)

The safety of three different kinds of rural highways in Texas—two-lane without paved shoulders, two-lane with paved shoulders, and undivided four-lane without paved shoulders—were reported. Both a comparative analysis and a before and after technique were used to determine the safety benefits. The results are shown in figure 3. (*16*)

Accident data were collected from 1975 through 1977 for 85 sites. A total of 16,334 accidents were included as the study base, 8,815 of these accidents were non-intersection accidents. Based on this study, the researchers concluded that two-lane roads with shoulders were safer than four-lane roadways without shoulders (termed "poor boys"). When paved shoulders are converted to travel lanes, the immediate recovery zone is removed and fixed objects are nearer the traveled lane. If safety is a major consideration, only "poor boy" highways should be considered for average daily traffic (ADT) volumes above 7,500. (*16*)

For 40 projects in Los Angeles, Calif., consisting of 50.7 km (31.5 miles) of arterial street widening, the analysis of 4,035 reported accidents showed that total accidents were reduced by 21 percent and injury plus fatal accidents were reduced by 22 percent. The primary purpose for the street widening was to increase capacity. Most streets were widened to have two lanes in each direction with continuous median or left turn channelization with parking allowed. Abrupt changes in the pavement width across a few lots along a street were eliminated to provide standard widths on 13 major secondary streets. The accident analysis consisted of counting the number of accidents for 24 months before and 24 months after the widening. The analysis was based on the actual number of accidents and not the accident rates even though traffic volumes increased during the after periods because of the increased capacity from widening. (*17*)

## Bridge width

Safety on bridges has been shown to be related to the width of the bridge and the width of the roadway approach (travelway plus the shoulder). (*18*) Table 2 compares accident rates and the related bridge and roadway width differences.

**Table 2.—Safety of narrow bridges (*18*, *19*)**

| Bridge width minus roadway width | Accidents per 100 million vehicles |
|---|---|
| *Feet* | |
| -6 | 120 |
| -4 | 103 |
| -2 | 87 |
| 0 | 72 |
| 2 | 58 |
| 4 | 44 |
| 6 | 31 |
| 8 | 20 |
| 10 | 12 |
| 12 | 7 |

1 ft=0.3 m

From the information contained in table 2, the expected effectiveness from bridge widening is shown in figure 4. (*19*)

Remedial treatment on the approaches to more than 50 narrow bridges (7.9 m [26 ft] wide or less) reduced the number of accidents on these bridges from 20 in a 22-month period to 4 in a 17-month period while the ADT increased from 4,780 to 5,690. (*20*)

## Medians

Research on a variety of medians in Kentucky has shown that both the total median accident rate and the accident severity rate decline with increasing median width. A breaking point or "leveling off" seems to occur for median widths between 9.1 and 12.2 m (30 and 40 ft) (fig. 5). (*21*)

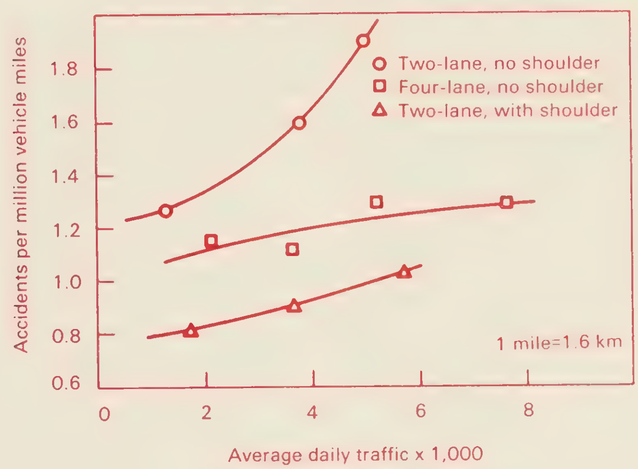


*Figure 3.—Accident rates for different classes of Texas highways. (16)*

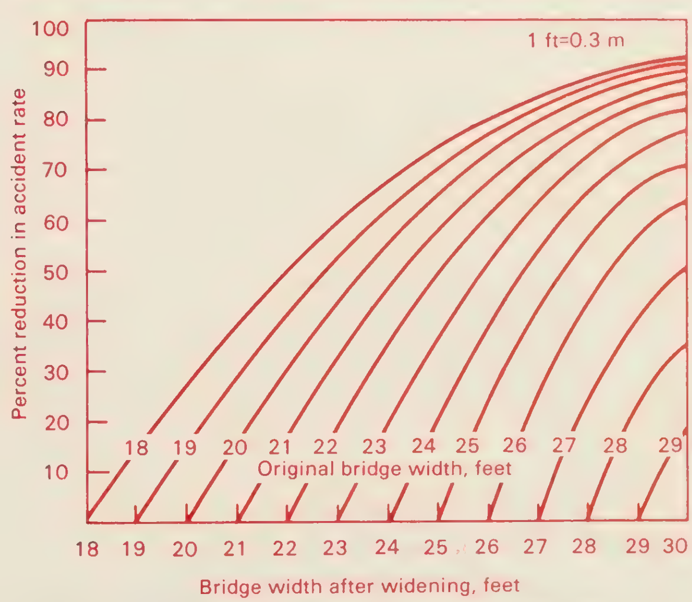


*Figure 4.—Percent reduction in accident rate associated with increases in bridge width. (19)*

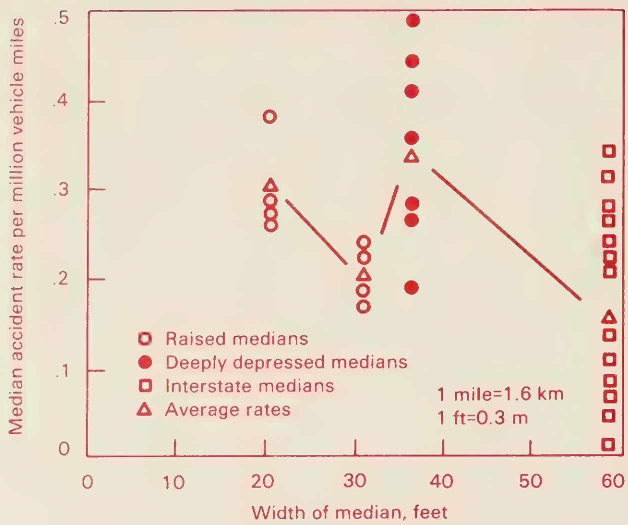


*Figure 5.—Median accident rate versus median width. (21)*

Other elements of the median, however, such as cross slopes and presence of obstructions and irregularities, can have a greater effect on safety of the median than width. The beneficial effect of wide medians can be completely negated by steep slopes. The steep slopes do not provide reasonable recovery areas and are often a hazard in themselves. (*21*)

A study of the safety benefits associated with 25.6 m (84 ft) wide medians as to mound type (raised) versus swale type (depressed) for interstate highways in Ohio indicated that either kind provides a generally adequate recovery area for encroaching vehicles, although the swale median appears to provide more opportunity for encroaching vehicles to regain control and return to their roadway. (*22*)

### Pavement cross slope

Roadway pavements generally are designed to slope from the centerline toward the edges to accommodate drainage during wet weather. Flat cross slopes on flat roadway sections cause water to accumulate on the pavement surface during heavy rains, and vehicles are more likely to be involved in hydroplaning accidents. A study in Louisiana, which gets 1 524 mm (60 in) of rainfall a year, found that roadways with relatively flat cross slopes are more accident prone than those with steeper slopes. (*4*)

The most critical location for hydroplaning is sag-vertical curves where the bottom of the curve is subject to flooding. Pavement cross slope is a dominant factor in removing water from the pavement surface and a minimum cross slope of 2 percent was recommended as a remedial treatment for these locations. (*23*)

### Side slope and ditches

The Highway Vehicle Object Simulation Model (HVOSM) evaluated the effects various side slopes and ditch configurations would have on vehicles that run off the road. The simulation results later were verified with field studies. The hinge point, the location where the shoulder meets the side slope, produced no critical adverse effects for side slopes of 3:1 to 10:1. (*24*)

Return maneuvers can be accomplished without vehicle rollovers on smooth firm embankments of 3:1 or flatter at speeds up to 129 km/h (80 mph) and encroachment angles of 15 degrees. However, to permit recovery, a coefficient of friction of 0.6 must be available. Almost no returns can be executed when the coefficient of friction is as low as 0.2 (a more probable value than 0.6).

The trapezoidal ditch configuration represents the most desirable cross section from a safety standpoint, particularly for ditches wider than 2.4 m (8 ft). The use of front slopes (the slope from the shoulder to the bottom of the ditch) steeper than 4:1 is not desirable because their use severely limits the choice of back slopes to produce a safe ditch configuration. Ditch evaluation curves for roadside slope combinations are shown in figure 6.

## Alinement

The alinement of a roadway includes both the horizontal line of the roadway, such as straight sections (tangents) and curves, as well as the vertical line (profile), such as grades and vertical curves (sags and crests). An estimate of the total number of accidents occurring on these various alinement features for 1979 and 1980 is presented in table 3. (*25*)

Although most accidents occur on straight, level ground, the curves and grades are more hazardous because accident rates are higher for these features, as will be shown in the following sections.

### Horizontal curves

Past research generally shows that as the degree of curve increases, the accident rate increases. (The degree of curvature is the central angle subtended by an arc of 30 m [100 ft].) In a classic study in 1953, 15 States provided information covering 1 year's accident experience (16,421 accidents) on about 8 000 km (5,000 miles) of highway. Factors studied included number of lanes, ADT, degree of curvature, pavement and shoulder widths, frequencies of curves, and other sight-distance restrictions. Two-lane, three-lane, and four-lane divided and undivided roadways were analyzed. Adjustments

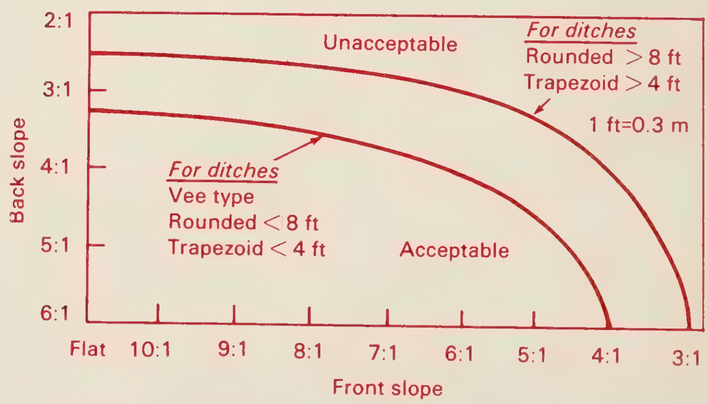


*Figure 6.—Ditch evaluation for roadside slope combinations. (24)*

Table 3.—Percent of total accidents by alinement and profile features in the United States for 1979 and 1980 (total number of accidents = 6,773,000) (25)

| Vertical alinement | Single-vehicle accidents | | Multivehicle accidents | | Total |
|---|---|---|---|---|---|
| | Curved | Straight | Curved | Straight | |
| On grades | 3.0 | 3.6 | 1.5 | 7.7 | 15.8 |
| Sag or hillcrest | 0.6 | 0.6 | 0.4 | 0.8 | 2.0 |
| Level ground | 3.9 | 23.2 | 3.7 | 51.4 | 82.2 |
| Total | 7.1 | 27.4 | 5.6 | 59.9 | 100.0 |

were considered for differing reporting requirements but the analyses using unadjusted accident rates appeared to be most reasonable. For all kinds of roadways, the sharper the curve, the higher the accident rates. (*26*)

Accident records for the Pennsylvania and Ohio Turnpikes were studied. The effects of horizontal and vertical alinement on accident rates were analyzed. On the Ohio Turnpike, no significant accident dependence on either grade or curvature was found, except that a 1° curve on a 3-percent downgrade had a very high accident rate. The data showed that the Pennsylvania Turnpike accident rate was not dependent on grade, but it does increase with increasing curvature. (*27*)

Three accident data bases—Skid Reduction, Delineation, and Calspan—were analyzed to quantify safety performance of rural two-lane highways.[2]

The Skid Reduction data base consisted of accident rates, skid numbers, ADT's, and related geometric data for two 1-year periods on 455 sections of two-lane rural highways, including 3 560 km (2,212 miles) in 15 States. Before accidents totaling 7,157 were for 1-year periods during 1970 through 1973. The 1-year-after period was during 1974 and/or 1975 for 363 of the 455 sections, which included 2 829 km (1,758 miles) with 6,032 accidents.

The Delineation data base contained data on accidents, geometrics, traffic controls, and traffic volumes on 320 roadway sections and 194 horizontal curves. The 320 roadway sections had 12,414 recorded accidents, and the curves had 5,022 accidents.

The Calspan data base provides 6,651 single-vehicle accidents that occurred on two-lane rural roads during 1975 through 1977 in six States. The data provided 375 items, such as roadway geometric data and traffic volume, for each accident rather than the number of accidents that occurred on specific roadway sections. Thus, accident rates as such were not recorded.

Table 4 shows that accident rates generally increase with the degree of horizontal curvature for the Delineation base but have no apparent trend in the Skid Reduction data base. The accident rate for tangents for the Skid Reduction data base is slightly less than the overall accident rate for horizontal curves. The accident rates for the Delineation data base are significantly higher than for the curves in the Skid Reduction data base. The Delineation data base may contain a bias because of the selection of horizontal curves to receive delineation treatment on the basis of high accident experience.

The Calspan data base provides insight into the characteristics of single-vehicle accidents on two-lane rural curved versus tangent highways. Table 5 analyzes single-vehicle accidents by route familiarity by the driver. Although 23 percent of drivers who had accidents on tangents traveled the roadway rarely or for the first time, 31 percent of the accidents on curves involved such drivers. Although curve warning devices may not have an effect on the local driver, there may be substantial benefits to drivers who rarely frequent that section of highway.

Departure direction for right and left curves by degree of curvature also was developed. The percentage of departures on the outside of curves increases as the degree of curvature increases for both right and left curves. For tangents, vehicles are twice as likely to depart on the right as on the left, presumably because the vehicles are closer to the right-hand roadside and have more recovery room on the left or may become involved in a multivehicle accident before departing on the left.

Table 4.—Accident rates for tangents and horizontal curves for two-lane rural roads

| Horizontal curvature | Accidents per million vehicle miles: Skid Reduction data base | Delineation data base |
|---|---|---|
| *Degrees* | | |
| 0 (tangent) | 2.199 | |
| Less than 1.55 | 2.252 | |
| 1.55 - 3.25 | 2.503 | 4.590 |
| 3.25 - 5.50 | 2.319 | 5.960 |
| Over 5.50 | -- | 7.718 |
| All degrees | 2.329 | 6.196 |

1 mile=1.6 km

**Transition curves and superelevation**

Transition curves often are used to connect the tangent section of the roadway with the horizontal curvature of a roadway. For curves with long radii (small degrees of curvature) transition curves often are not used. For sharper curves, either compound curves (two simple curves are used to connect with the main curve) or a spiral curve (a curve with an increasing radius from the tangent to the main curve) is used to connect with the tangent section. Although no accident studies have been reported on the application of transition curves, a study in 1980 analyzed the vehicle dynamics for transition curves. (*28*) The HVOSM was used for the analysis of 8°, 31°, and 38° curves. A vehicle's lateral acceleration with no transition curve was as much as 50 percent greater than the steady-state acceleration; the spiral transition simulated less than 10 percent. It was concluded that no transition was the worst case, compound transitions were better, and the spiral allowed the easiest path to follow. Superelevation effects for the three curves were also studied. Superelevation does not appear to play a significant role

[2]S. A. Smith, J. E. Purdy, et al., "Identification, Quantification, and Structuring of Two-Lane Rural Highway Safety Problems and Solutions, Vol. III—Physical and Safety Characteristics of Two-Lane Rural Highways," unpublished appendix, JHK and Associates, Inc., Alexandria, Va., 1981.

**Table 5.—Single vehicle accidents by horizontal alinement by route familiarity**

| Familiarity | Tangent | | Left curve | | Right curve | |
|---|---|---|---|---|---|---|
| | Number of accidents | Percent | Number of accidents | Percent | Number of accidents | Percent |
| Daily | 1,066 | 35.1 | 425 | 29.0 | 276 | 30.2 |
| 1+ week | 786 | 25.9 | 375 | 25.6 | 211 | 23.1 |
| 1+ month | 472 | 15.6 | 206 | 14.0 | 146 | 16.0 |
| Rarely | 464 | 15.3 | 281 | 19.2 | 165 | 18.0 |
| First time | 245 | 8.1 | 180 | 12.2 | 117 | 12.8 |
| Total | 3,033 | 100.0 | 1,467 | 100.0 | 915 | 100.0 |

in affecting transient vehicle dynamics on curves but does influence the steady-state steer characteristics of the vehicle. The greater the superelevation, the less the vehicle understeers.

Causes for a high accident rate (55 accidents in 6 years) on a 1° curve and 2-percent downgrade on the Ohio Turnpike were studied in detail. (*27*) Of the total accidents, 67 percent involved skidding during wet weather. The lateral acceleration for this curve is relatively large and the water depth during rainstorms would be greater than on smaller radius curves having larger superelevations. It was recommended that on long radius curves, higher superelevations be used to compensate for the increased drainage path length. Increasing the superelevation will reduce the water depths and thus reduce the wet weather accident rate.

**Delineation treatments**

One countermeasure to make horizontal curves safer involves delineation treatments along the roadway. Accidents on two-lane highways that might possibly be related to delineation treatments or the lack thereof were reviewed. (*29*) The Delineation data base was used. Delineation-related accidents were considered to be those accidents that did not involve snowy or icy pavements, collisions with an object on the pavement, defective roadways or vehicles, or improper maneuvers on roadways. It was determined from the data base that the portion of delineation-related accidents for three alinement situations was tangent—68 percent, winding roads—80 percent, and horizontal curves—74 percent.

Eighteen delineation treatments were tested ranging from just a solid painted centerline to the centerline marked with retroreflective raised pavement markers, edgelines with raised pavement markers, and postmounted delineators. These treatments were compared with the standard centerline and edgeline treatments. An accident potential model was developed and validated to test the various treatments. This model involved measurements of vehicles centrally indexed (CI), the mean location of the vehicles in relation to the center of the lane, and the difference in lateral placement variance (DPV), which is the variance of the location of the vehicles within the lane divided by the lane width. The accident rate (AR) is the number of nighttime, delineation-related, nonintersection accidents per million vehicle miles (dry pavement conditions).

$$AR = -0.22 + 1.15\ CI + 25.3\ DPV$$

The 18 treatments were applied to eight test sites (four tangent sections, two winding roadways, and two horizontal curves) and measurements of speed as well as CI and DPV were made to estimate the safety potential of the various treatments. The study concluded that several less paint-intensive delineation systems performed as well or better than the more expensive base condition. This included the centerline skip ratio of 3 m (10 ft) paint stripes and 9 m (30 ft) gaps rather than the 4.6 m (15 ft) and 7.6 m (25 ft) spacings. This provided an estimated 4 percent cost savings. It was recommended that edgelines from 51 to 76 mm (2 to 3 in) wide could also be used. This would save costs of an additional 12 percent, if adopted.

Where severe visibility conditions occur from frequent fog or blowing sand, it was recommended that retroreflective raised pavement markers be considered at 24.4 m (80 ft) intervals where passing is permitted and 12.2 m (40 ft) intervals where passing is prohibited. Where raised pavement centerline markers cannot be applied because of snowplowing, postmounted delineators should be installed at 122 to 161 m (400 to 528 ft) intervals on tangents and the Manual on Uniform Traffic Control Devices recommended spacing for curves of various radii. (*3*) The postmounted delineators on tangents had a negligible effect on lateral placement but did affect speeds and placement variance on curved sections.

Further research is needed on the use of a single solid centerline in no-passing zones. The use of reflective raised pavement markers on both the centerline and edgeline reduces potential hazard by 68 percent but costs about 900 times as much as the standard painted markings and was considered to be too expensive for general use.

Eight States participated in studies to evaluate 13 kinds of postmounted delineators.[3] Although flexible posts cost twice as much as the standard U-channel kind, it would be cost effective to use the flexible posts in areas subject to numerous impacts if the flexible posts can survive two or more hits. The accident data collected by the States indicate a trend toward reducing run-off-the-road accidents where postmounted delineators are installed. The Montana Department of Highways reported a 30-percent reduction of run-off-the-road nighttime accidents at delineator test locations (primarily curve and narrow bridge sections) with the large delineators being more effective than the small ones. (*30*)

**Curve warning signs**

A study conducted at two sites on the Maine Facility examined the effectiveness of several alternative sign configurations (both warning and regulatory) for warning motorists of a hazardous horizontal curve ahead in a rural two-lane situation. (*31*) Despite relatively large decreases in speed in the vicinity of the curve, no sign configuration was found to be consistently more effective in reducing speeds than another. The study did not test, however, the curve sites without the standard curve warning sign. The sign configurations included the standard curve warning signs alone, with advisory speed plates, with "Reduced Speed Ahead" sign and a regulatory speed limit sign, and with a "Maximum Safe Speed 35 MPH" sign. Also, a test condition involved changing the placement of the curve warning signs in advance of the curve from 152 to 213 m (500 to 700 ft).

**Grades**

The vertical alinement of a highway is a combination of straight roadway sections at a set slope (grades are expressed as the percentages of rise to horizontal distance) and vertical curves (usually parabolic) to connect the slopes in crest or sag curves.

It was found that grade alone did not have any particular effect on accident rates for tangent sections on any kind of rural highway. (*26*) The combination of grade and horizontal curvature did show steeper grades increase the accident rates for two-lane rural curved sections with ADT's between 5,000 and 9,900 vehicles per day.

The safety effects of long, steep grades on two-lane rural highways were analyzed using a computer simulation model to determine traffic speed distributions. (*32*) Accident rates were then estimated using accident involvements presented as a function of the deviation from the mean speed. (*33*) Accident estimates were made for a variety of terrains.

**Table 6.—Accident rates related to grades on German expressways (*34*)**

| Grade | Accidents per million vehicle kilometres |
|---|---|
| *Percent* | |
| 0 - 1.9 | 0.46 |
| 2 - 3.9 | 0.67 |
| 4 - 5.9 | 1.90 |
| 6 - 8 | 2.10 |

Accident rates on German expressways were studied. (*34*) Accident rate increased as the grade increased as is shown in table 6. The table shows steep grades of 6 to 8 percent produce over four times the accidents as gradients under 2 percent.

Also, the combination of grades and horizontal curvature was at high accident locations. The superelevation in combination with the grade produced oblique gradients of over 8 percent. Skidding accidents occurring during wet weather caused vehicles to either slide off the road or collide with vehicles they were passing. As the grades became steeper and the degree of curvature increased, the accident rate increased.

**Vertical curves**

Vertical curves are installed to connect grades of different slopes. The lengths of the vertical curves are usually based upon the difference between the grades and the required stopping sight distance for the design speed of the roadway. For crest vertical curves, the sight distances are determined for drivers to see over the top of the hill to objects on the other side. For sag vertical curves the sight distances are determined for drivers seeing at night from the vehicles' headlights. The crest vertical curve is one of the primary features of the roadway that limits sight distance.

An analysis of sight distance restrictions on tangents was conducted for two-lane rural roads. (*26*) For the study, a sight distance of less than 183 m (600 ft) for flat or rolling terrain or less than 122 m (400 ft) for mountainous terrain was considered to be a sight restriction. The accident rate increased as the restriction frequency increased from zero to about two restrictions per kilometre (about three per mile).

About 10,000 accidents on 86.9 km (54 miles) of freeways in the five largest Texas cities were investigated. (*35*) A concentration of accidents at crest and sag vertical curves was found. Rear end collisions resulting from following too closely comprised 70 percent of all accidents. The study showed unfavorable sight conditions were present at the high accident frequency crest and sag locations.

[3]C. W. Niessner, "Postmounted Delineators," *Federal Highway Administration*, Washington, D.C. Not yet published.

### Limited sight distance controls

Limited sight distance warning signs in New York were evaluated. (*36*) Spot speed studies were taken at 14 locations in five counties with and without the warning sign and its accompanying advisory speed panel. Speeds were recorded at the crest of the vertical curve. The results indicate the warning signs with advisory speed panels had no effect in slowing the speed of vehicles; in fact, at five sites, the speeds decreased when the signs were removed.

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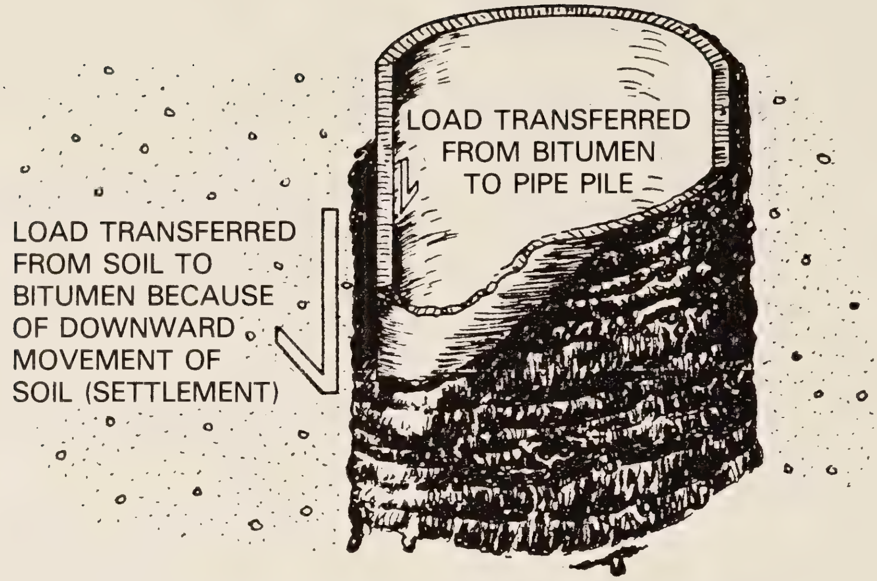



BITUMEN LAYER MINIMIZES TRANSFER OF DOWNDRAG THROUGH ITS DEFORMATION PROPERTIES

# Bitumen Coating on Piles to Reduce Downdrag

by
**George Machan and L. Radley Squier**

## Introduction

Embankments constructed over compressible soil deposits cause the ground to settle, with the settlement occurring at a decreasing rate for many years. Pile foundations constructed in or near the embankments are exposed to the settling ground, which induces negative skin friction forces, or downdrag loads, on the piles. Often, thick deposits of compressible soils cause unacceptably high downdrag loads and, therefore, the pile design requires some method of reducing downdrag.

The Mocks Bottom overcrossing, near Swan Island in Portland, Oreg., is underlain by a relatively thick compressible clayey silt deposit. The overcrossing is a three-span bridge with 7.6 m (25 ft) high approach embankments. The two interior piers and the abutments of the bridge are supported by 324 mm (12¾ in) diameter pipe piles. The magnitude of the estimated downdrag on uncoated pipe piles, as a result of ground settlements, was unacceptable and, therefore, the piles were coated with bitumen to reduce the downdrag to acceptable levels.

This article describes the project and presents the predictions of downdrag loads on both uncoated and bitumen-coated pipe piles, and describes the use of a unique pile uplift test to evaluate the anticipated downdrag on the bitumen-coated production piles.

## Design Considerations

The bridge site is mantled by 3.0 to 6.1 m (10 to 20 ft) of relatively compact hydraulic fill (silts and sands), underlain by an 18.3 to 21.3 m (60 to 70 ft) thick, soft to medium stiff gray clayey silt deposit. The gray clayey silt contains varying amounts of clay and fine organics. Properties of the clayey silt are listed in table 1. Underlying the clayey silt is a medium dense to dense fine sand with a very dense gravel stratum about 42.7 m (140 ft) deep. The groundwater table is about 6.1 m (20 ft) below the ground surface. A geologic profile is presented in figure 1.

The placement of the approach fills would have resulted in relatively large settlements occurring over decades. For example, it was estimated that the 7.6 m (25 ft) high approach fill would settle about 1.1 m (3½ ft) (primary consolidation) in about 8 years, with an additional 127 mm (5 in) of settlement (secondary consideration) over the following 50 years.

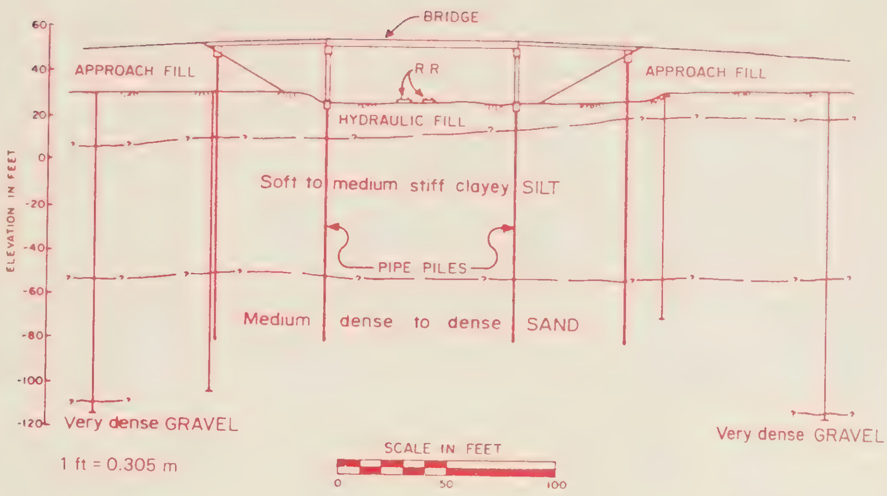



*Figure 1.—Geologic profile.*

To insure that primary consolidation settlements would occur during the 1-year construction period, prefabricated vertical drain wicks (Alidrains) were installed on a 3.0 m (10 ft) center-to-center triangular array to accelerate ground settlements. Each drain was about 27.4 m (90 ft) long to totally penetrate the gray clayey silt deposit. A 2.4 m (8 ft) surcharge fill was placed to restore site grade from ground settlements and to reduce the amount of postconstruction settlements.

The bridge is supported on concrete-filled 324 mm (12¾ in) diameter steel pipe piles with a wall thickness of 10 mm (⅜ in). The pipe piles were driven closed-ended into the dense sands that underlie the gray clayey silt stratum. Design pile capacity is 91 Mg (100 tons).

The maximum downdrag was estimated for uncoated pipe piles for two different conditions: The abutments in the new embankment fill about 7.6 m (25 ft) high (bents 1 and 4) and at the intermediate bents (2 and 3) about 3.0 to 6.1 m (10 to 20 ft) from the toe of the embankments (no new fill placed). Table 2 presents estimated downdrag loads using various analytical methods. (*1–4*)[1]

[1]Italic numbers in parentheses identify references on page 151.

The calculated magnitudes of downdrag exceeded the design capacity of the piles. Methods investigated for reducing the downdrag included:

- Increasing the length of the bridge, thereby reducing thickness of approach fills and the resulting adverse settlement.
- Using isolation-type casing or bentonite slurry in predrilled holes.
- Increasing the number of piles and increasing the pile capacity.
- Using a bitumen coating on the foundation piles.

Benefit/cost studies of each of these methods indicated that using a bitumen layer on the foundation piles would be the most cost effective.

## Bitumen-Coated Piles to Reduce Downdrag

Most case histories involving bitumen-coated piles are from Europe, with few reported from the United States. The literature indicates that by using bitumen, downdrag typically is reduced 90 to 98 percent. However, one case history reports a reduction of 30 to 80 percent (*5*), while another reports a reduction of 60 to 80 percent. (*6*) Both cases used relatively thin (1 to 3 mm [0.04 to 0.12 in]) bitumen coats. Basically, the effectiveness of a bitumen coating appears to depend on the viscoelastic properties of the bitumen at ground temperature, the thickness of the bitumen coating, and the rate of ground settlement.

The appropriate thickness of bitumen coating has not been agreed upon. On one hand, a relatively thin coat (1 to 2 mm [0.04 to 0.08 in]), unless the soil and settlement conditions are extreme, would only require a small amount of bitumen and is less likely to flow in storage and to peel off during driving. (*7, 8*) On the other hand, thin coats may

**Table 1.—Properties of gray clayey silt**

| | |
|---|---|
| Compression index, $C_c$[1] | 0.35 to 0.60 |
| Average coefficient of consolidation, $c_v$[1] | 0.35 ft²/day |
| Coefficient of secondary compression, $C\alpha$[1] | 0.75 to 1.0 percent per log cycle time |
| Atterberg limits, PL | 40 percent |
| LL | 56 to 71 percent |
| Moisture contents, w | 40 to 75 percent |
| Shear strength, c[2] | 0.3 to 0.5 ton/ft² |

[1]Determined from laboratory consolidation tests. 1 ft² = 0.09 m²
[2]Determined using Torvane shear device on undisturbed samples. 1 ton/ft² = 9.8 Mg/m²

**Table 2.—Estimated downdrag load on uncoated piles**

| Design method | Estimated downdrag Bents 1 and 4 | Bents 2 and 3 |
|---|---|---|
| | *Tons* | *Tons* |
| Beta method (*1*) | 240 | 110 |
| Average shear strength (*2*) | 145 | 90 |
| Effective stress approach (*3*) | 220 | 100 |
| Effective stress method (*4*) | 255 | 105 |

1 ton = 0.91 Mg

**Table 3.—Prediction of downdrag on bitumen-coated piles**

| Type | Bitumen properties: Penetration at 25° C | Bitumen properties: Softening point | Estimated bitumen viscosity | Estimated downdrag: Bents 1 and 4 | Estimated downdrag: Bents 2 and 3 |
|---|---|---|---|---|---|
| | *Centimetres* | *Degrees Fahrenheit* | *N-sec/m²* | *Tons* | *Tons* |
| AASHTO M 115 Type C | 22 | 187 | $2 \times 10^7$ | 8 | 3 |
| AASHTO M 190 (culvert compound) | 72 | 173 | $2 \times 10^6$ | 1 | 1 |
| ASTM D 2521 (canal liner) | 62 | 184 | $2 \times 10^6$ | 1 | .50 |

°C = (°F−32)/1.8
1 ton = 0.91 Mg

not perform as anticipated in the long term because of oxidation and contamination of the bitumen layer. Therefore, the minimum coating thickness should be 6 mm (¼ in) with 10 mm (⅜ in) optimum—a much higher degree of assurance can be obtained at a relatively small increase in materials cost. The cost of applying the bitumen coating is about 10 to 20 percent of the pile foundation costs. (*9*)

The following expression illustrates the relationship between certain physical parameters and the magnitude of downdrag (negative shear stress). (*5, 9*)

*equation 1:*

$$\sigma_{neg} = \eta \times \frac{\varepsilon_r}{t}$$

Where,

$\sigma_{neg}$ = Negative shear stress
$\eta$ = Bitumen viscosity
$\varepsilon_r$ = Strain rate
t = Bitumen thickness

In a similar manner, a relationship for estimating downdrag (negative shear stress) on bitumen-coated piles has been expressed. (*10*)

*equation 2:*

$$\sigma_{neg} = m \left[\frac{\Delta_{avg} \div t}{S_r}\right]^n$$

Where,
m and n = Bitumen parameters determined at the anticipated ground temperature
$\Delta_{avg}$ =Average ground settlement rate
$s_r$ =Reference shear rate

The reference shear rate arbitrarily chosen was $10^{-5}$ $sec^{-1}$.

The strain rate, or the ground settlement rate, is estimated using consolidation theory. For deep piles, the zone of settlement may be subdivided and the average ground settlement rates computed. The negative shear stress is then computed for each of the pile segments. The estimated downdrag force on a pile segment is equal to the average negative shear stress × pile perimeter × length of pile segment. The estimated total downdrag force is the sum of the segments. According to these expressions, the negative shear stress on a bitumen-coated pile can be reduced by selecting a softer bitumen (lower viscosity) and/or increasing the thickness of the bitumen coating.

The viscosity of the bitumen may be determined directly by laboratory testing. However, if such testing is not available locally, the viscosity may be estimated (at the corresponding ground temperature) using a nomograph. (*9*) The nomograph presents a correlation between viscosity and the penetration at a known temperature, the softening point temperature (ring and ball), and the average ground temperature. (*11*)

Actual field tests can be performed or it can be assumed that a properly applied bitumen coat can reduce the negative skin friction to a value of about 976 kg/m² (200 lb/ft²). This value will in most cases give an adequate prediction of the downdrag load and allow for rational pile design. (*8*)

## Predictions of Downdrag

For the two downdrag conditions previously described (bents 1 and 4 and bents 2 and 3), the estimated downdrag per pile (using equation 1) is presented in table 3 as a function of several bitumen types. The average coating thickness specified is 10 mm (⅜ in). (*9*) The ground temperature was assumed to be 10° C (50° F). The ground profile consisted of hydraulic fill overlying the gray clayey silt and, at the abutments, embankment fill. The embankment and hydraulic fills were assumed to settle the same rate as the ground surface. The settlement rate of the clayey silt was taken as an average of the total settlement (ground surface settlement rate ÷ 2). The maximum settlement rates of the fills were expected to be 13 mm and 6 mm (½ in and ¼ in) per day at bents 1 and 4 and bents 2 and 3, respectively (fig. 1), after the piles were installed. Using a negative skin friction of 976 kg/m² (200 lb/ft²) (*8*), downdrag loads of 33.6 Mg and 25.4 Mg (37 tons and 28 tons) were computed for bents 1 and 4 and bents 2 and 3, respectively. If this skin friction value is associated with the thin bitumen coats, then it is probable that the downdrag estimate would be less for thicker bitumen coats.

## Uplift Tests

To refine the design, uplift tests were performed on two bitumen-coated test piles to evaluate application methods and the integrity of the bitumen after driving and to provide a field-measured relationship between pile load and strain rate. The test piles used were PP 12 ¾ × 0.375 steel pipe piles, 25.0 m (82 ft) long, with conical steel tips. The uplift tests were conducted before the embankment was placed; hence, the lengths of the test piles corresponded to the combined thickness of hydraulic fill and gray clayey silt at bent 2. Each pile was made up of two 12.2 m (40 ft) long sections, spliced together in the leads. The piles received a thin coat of bitumenastic primer and, subsequently, a 10 mm (⅜ in) (nominal) coat of bitumen. Culvert compound (AASHTO M 190–70) bitumen was used to coat test pile No. 1 and canal liner (ASTM D 2521–76) bitumen was used to coat pile No. 2. A 19 mm (¾ in) square bar was welded to the perimeter of the test piles at the tip to provide a collar that would create an annular space between soil and pile to protect the bitumen coating during pile driving. (*12*) Before pile driving, the bitumen coating was measured (34 measurements per pile) to confirm a nominal thickness of 10 mm (⅜ in) (standard deviation of ± 3 mm [⅛ in]).

The test piles were driven with a Vulcan 010 hammer (43.4 J [32,500 ft-lb] rated energy). Test pile No. 1 was driven conventionally, while test pile No. 2 was placed in a 6.1 m (20 ft) deep, 457 mm (18 in) diameter hole filled with a thick bentonite slurry and then driven to the specified tip elevation. A minimum 2-week delay was specified before performing uplift tests to allow for dissipation of excess porewater pressures from pile driving.

The guidelines used for the uplift tests were in general accordance with ASTM D 3689–78. The test frame consisted of steel beams with ends supported on timber cribbing. The arrangement of the test frame provided reaction for a 91 Mg (100 ton) hydraulic jack.

As uplift loads were applied, movement of the top of the pile was measured using two dial gages mounted equidistant from the pile center and on opposite sides of the pile. The dial gages measured the upward movement of each pile with respect to stationary reference beams that were supported on timber cribbing a minimum of 2.1 m (7 ft) beyond the reaction cribbing. Secondary measurement systems consisted of wire, mirror, and scale; and surveyor's level and steel scale (with an accuracy of 0.25 mm [0.01 in]).

Test pile No. 1 was pulled upward in 4.5 Mg (5 ton) load increments, up to 45 Mg (50 tons), then in 9 Mg (10 ton) increments to 64 Mg (70 tons). The 91 Mg (100 ton) jack suffered a hydraulic failure at 64 Mg (70 tons) and the test was terminated. Test pile No. 2 was pulled upward in 9 Mg (10 ton) increments up to 54 Mg (60 tons); an ultimate pullout load of 58 Mg (64 tons) was measured. After the uplift tests were completed, the test piles were extracted using jetting techniques. An examination of the extracted test piles showed that most, if not all, of the bitumen coating remained on the pile. The nominal bitumen thickness was 10 mm (⅜ in), based on 34 tests per pile. The bitumen was not significantly affected by soil grains (sand, silt, clayey silt), and contamination of the bitumen was only surficial.

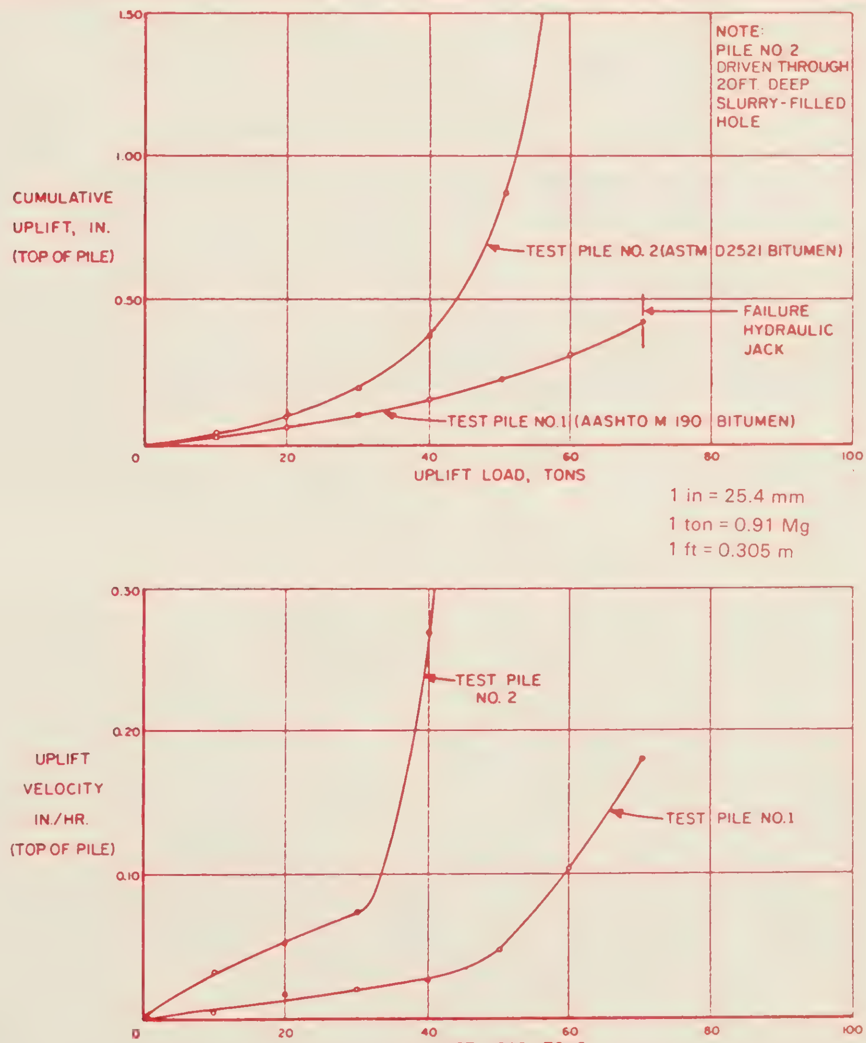



*Figure 2.—Results of pile uplift tests.*

The results of the uplift tests are presented in figures 2 and 3. Figure 2 shows the relationships between cumulative uplift movement versus applied load and between uplift velocity versus applied load. A unique relationship was observed for each load increment—a constant uplift velocity (strain rate) occurred after the initial pile elongation/uplift for test pile No. 2 (fig. 3). Using the uplift loads and the corresponding strain rates (velocities) the apparent viscosity of the bitumen coating was calculated for each test pile, using equation 1. (In the analyses, the upper 6.1 m (20 ft) of test pile No. 2, which was installed in a slurry-filled hole, was assumed not to contribute to skin friction and was thus excluded from consideration.) Then, using the same equation and the predicted ground settlement rates, the downdrag loads were calculated for the production piles. Table 4 presents the results of the calculations.

The shear stress, $f_s$, acting along each test pile was back-calculated from the uplift test data for various velocities (strain rates). For test pile No. 1, for example, coated with AASHTO M 190 bitumen, the back-calculated shear stresses were 1 109, 542, and 271 kg/m² (225, 110, and 55 lb/ft²) for strain rates of 13, 6, and 3 mm (½, ¼, and ⅛ in) per day, respectively. On the other hand, shear stresses for the test pile No. 2 with ASTM D 2521 bitumen were 281, 143, and 69 kg/m² (57, 29, and 14 lb/ft²) for strain rates of 13, 6, and 3 mm (½, ¼, and ⅛ in) per day. Using these shear stresses and the predicted settlement rates, downdrag loads were predicted, as shown in table 5.

Regardless of the method of analysis, it appears that production piles provided with ASTM D 2521 (canal liner) would have downdrag loads estimated to be 9 Mg (10 tons) (bents 1 and 4) and 4.5 Mg (5 tons) (bents 2 and 3). This constitutes about a 97 percent reduction of the estimated downdrag on an uncoated pile. For design, downdrag

**Table 4.—Estimated downdrag using back-calculated viscosity**

| Bitumen type | Back-calculated viscosity | Estimated downdrag Bents 1 and 4 | Bents 2 and 3 |
|---|---|---|---|
| | *N-sec/m²* | *Tons* | *Tons* |
| AASHTO M 190 (culvert compound) | 7.3 × 10⁷ | 30 | 10 |
| ASTM D 2521 (canal liner) | 1.8 × 10⁷ | 7 | 3 |

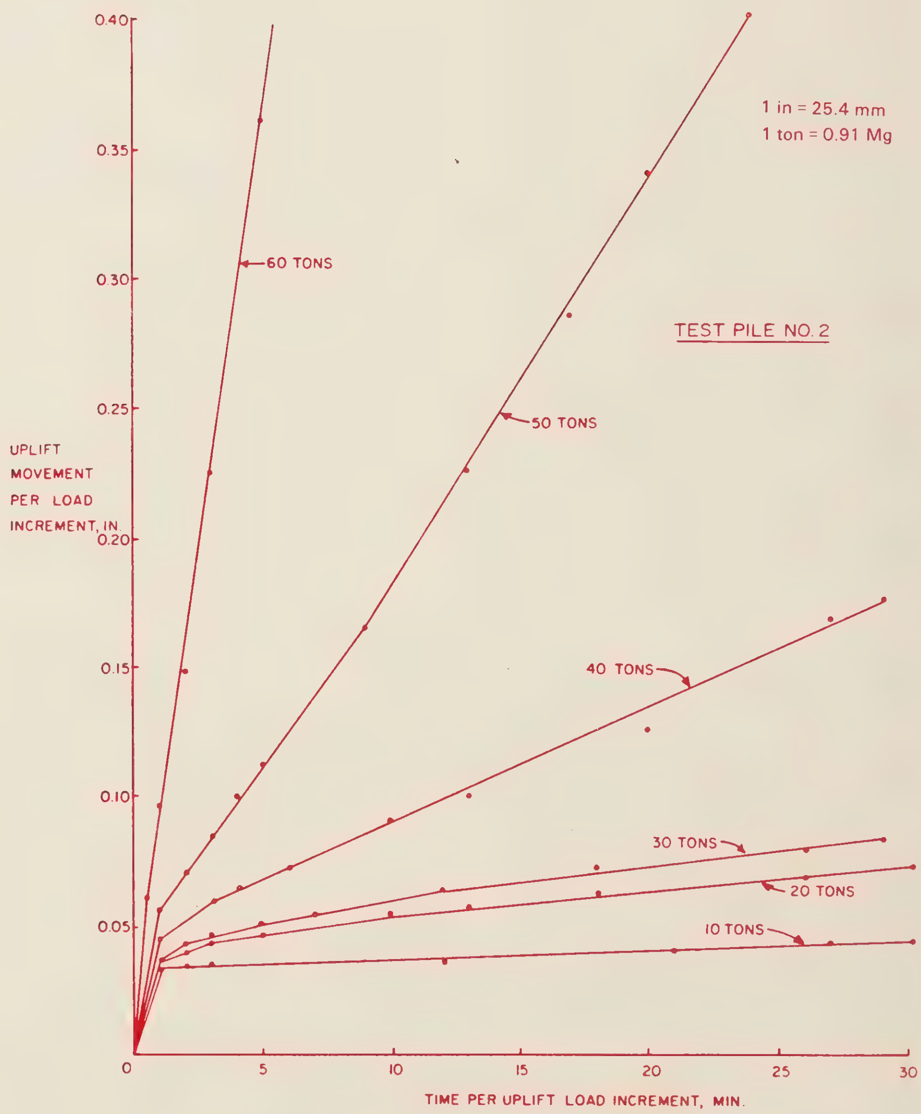
1 ton = 0.91 Mg

**Table 5.—Estimated downdrag using back-calculated shear stress**

| Bitumen type | Estimated downdrag Bents 1 and 4 | Bents 2 and 3 |
|---|---|---|
| | *Tons* | *Tons* |
| AASHTO M 190 (culvert compound) | 29 | 7 |
| ASTM D 2521 (canal liner) | 10 | 3 |

1 ton = 0.91 Mg

*Figure 3.—Results of uplift tests on test pile No. 2.*



loads of 27.2 Mg and 13.6 Mg (30 tons and 15 tons) were recommended for the abutments and intermediate piers, respectively, which provides for a factor of safety equal to 3. Instrumentation data from the actual bridge piles (to be described later) were expected to indicate the degree of conservatism adopted.

Predicted downdrag loads using equation 1 and the nomograph are about 5 to 30 times lower than those presented in tables 4 and 5. This large variance may be from the selection of realistic values of bitumen viscosity using the relationship in the nomograph. (*9, 11*) Also, using the value 976 kg/m$^2$ (200 lb/ft$^2$) (*8*), yields predicted downdrag loads of 33.6 Mg and 25.4 Mg (37 tons and 28 tons), which may be somewhat conservative, but not unreasonably so.

The instrumentation of the actual bridge piles was expected to indicate reasons for the variance between values of downdrag calculated from the pile uplift testing (values in tables 4 and 5) and use of viscosity from the nomograph combined with equation 1.

This pile instrumentation program was expected to increase the knowledge of the beneficial effects of bitumen on downdrag reduction and provide greater insight into the relationship between bitumen properties and ground settlement rate as related to magnitudes of downdrag load. Furthermore, the instrumentation data were expected to confirm that the downdrag can be predicted from uplift pile tests that measure uplift velocity (strain rate) versus applied load. The main features of the instrumentation program are discussed below.

## Instrumentation Program

The pile instrumentation program, which includes ground settlement monitoring, was designed to measure the downdrag loads on two production piles. The downdrag loads were correlated with ground settlement measured at various depths near one of the instrumented pipe piles. In addition, ground surface settlements were measured by periodic survey (coordinated with instrumentation monitoring).

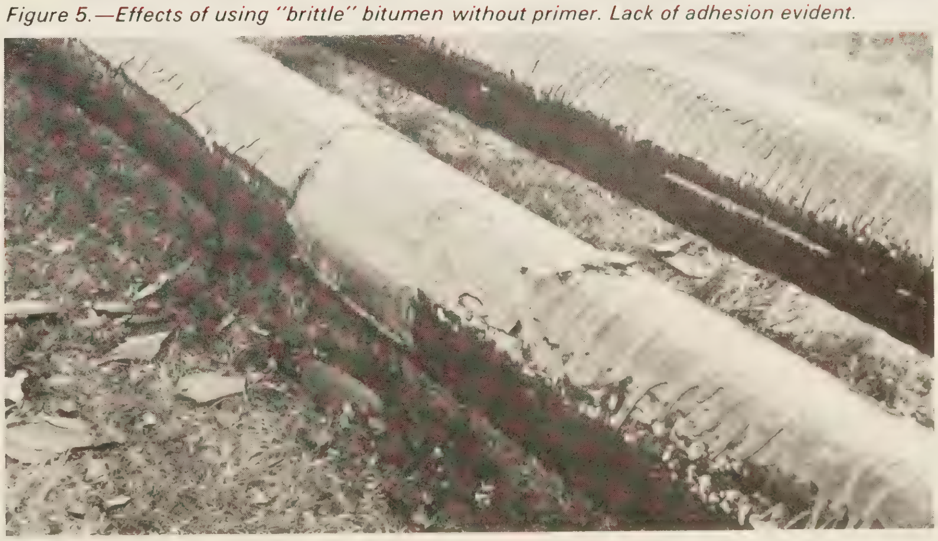
One of the production piles that was instrumented was located in the 7.6 m (25 ft) high embankment; the other one was located at an interior pier footing—between the toe of the embankment and the railroad. Instrumentation consisted of vibrating wire strain gages attached to a reference pipe that was inserted into the pipe pile and subsequently grouted. The pipe pile near the railroad was further instrumented with two mechanical extensometer telltales. Multipoint mechanical extensometers to provide a measurement of settlement of depth were installed in a nearby borehole. Double-acting hydraulic anchors were used to provide fixity at the appropriate depth for each of the extensometers.

### Construction considerations

The test piles were coated with bitumen in January 1981 when the outside temperature varied from −1° to 10° C (30° to 50° F). About 20 coats of a bitumen conforming to AASHTO M 115 Type C were applied using mops, to achieve a total thickness of 10 mm (⅜ in). A primer was not applied. At the low ambient temperature, the bitumen coating was hard and thus relatively brittle, increasing the possibility of bitumen spalling off the piles during driving. To evaluate this possibility, the pile was struck with a sledge hammer. The hammer blows cracked the bitumen and caused it to spall off the pile. Figures 4 and 5 show the damaged bitumen layer. A 1.5 m (5 ft) test section was then prepared

*Figure 4.—Unsatisfactory coating from using "brittle" bitumen and no primer.*

*Figure 5.—Effects of using "brittle" bitumen without primer. Lack of adhesion evident.*



using a thin coat of bitumenastic primer (conforming to AASHTO M 116). Figure 6 shows that the primer provided the required bonding; however, the brittle bitumen still cracked when hit with the hammer and some fragments dislodged. It was concluded that a softer bitumen that exhibited greater ductility/lower viscosity should be used.

With this in mind, a culvert compound (AASHTO M 190) and a canal liner bitumen (ASTM D 2521) were applied to primer-coated test piles No. 1 and No. 2, respectively. The 10 mm (3/8 in) coating was achieved in about seven applications. The piles subsequently were driven with essentially no loss of bitumen. Figure 7 shows the bottom 12.2 m (40 ft) pile section being lifted into the leads. The test pile was lifted with steel cables wrapped around the pile; this caused the bitumen to be scraped off locally. In the foreground of figure 7 are two buckets containing hot bitumen to be applied to the pile where the coating has been damaged.

*Figure 6.—"Brittle" bitumen combined with primer showed some adhesion.*

*Figure 7.—Lower section of test pile No. 2 being lifted into leads. Hot bitumen ready for repair of coating.*

Each test pile consisted of two 12.2 m (40 ft) sections, which were spliced by butt welding. Figure 8 shows the welding in progress. Care had to be exercised during welding to avoid igniting the volatiles in the bitumen (water was used to cool the piles).

After testing, it was difficult to extract the test piles so jetting techniques were used. Figure 9 shows the extracted piles with essentially no loss of bitumen other than that caused by cables and by transportation to the yard. The piles had a surface coating of clayey silt and sand; very little penetration of soil grains into the bitumen, if any, was observed.

The problems experienced in coating and handling the test piles led to different construction approaches for the production piles.[2] Two padeyes per pile section were welded to the piles to lift the piles, thus eliminating damage to the bitumen caused by cable wraps. The bitumen was heated to 149° C (300° F) and subsequently applied to the pile at a temperature between 93° and 149° C (200° and 300° F). (If the bitumen is too hot when applied, it melts the underlying coat and the required thickness cannot be obtained.) Also, the construction crew used a lathe to rotate (manually) the pipe piles for application of the bitumen (fig. 10). A trough is located beneath the pile to minimize waste and mess. Instead of mopping the bitumen on the production piles, the bitumen was poured using "watering" cans, which appeared to give excellent results.

*Figure 8.—Splicing test pile No. 1 in leads.*

[2]The bitumen-coating specification used for the steel production piles is reprinted at the end of this article. Also reprinted is a suggested specification for bitumen coating of concrete piles.

*Figure 9.—Extracted test piles showing localized cable-damaged coating.*

*Figure 10.—Production piles in lathe receiving bitumen coating.*

## Summary

The use of bitumen coating appears to reduce downdrag by about 90 to 97 percent (based on the results of the pile uplift tests). Based upon unit price bid items, the cost of the bitumen-coated piles was only about 15 percent greater than the cost of uncoated steel piles. Bitumen-coated piles are, therefore, a cost-effective method of reducing downdrag loads.

The downdrag load on a bitumen-coated pipe pile, predicted using bitumen viscosity (*11*), bitumen thickness, and strain rate, appears to be lower than the downdrag estimated from the pile uplift test results.

The instrumentation program implemented to monitor the production piles provided data for refining current prediction techniques. Of most interest is a confirmation that uplift load tests (measuring uplift velocity versus applied load) are a cost-effective way of obtaining data for pile design that considers downdrag. Also, it is hoped that the reasons for apparent discrepancies between theoretical expressions (equation 1) and actual downdrag load on piles will be better understood by evaluating the instrumentation program data.

The bitumen types that are more ductile (softer) may be better suited to cool climates because of their ability to sustain pile driving impacts at low ambient temperatures. Also, soft bitumens (high penetration/low viscosity) are better able to reduce downdrag. However, in some instances, design may require compromising between handling/storage/driving characteristics of the bitumen (given climatic conditions) and the desired viscosity properties in place.

## Acknowledgments

Foundation design and the pile uplift tests were conducted in conjunction with OBEC Consulting Engineers, Springfield, Oreg., who were responsible for structural design of the new overcrossing. The Project Manager for the Port of Portland is Carl V. Jonasson, Jr., and his assistance and support for the instrumentation program is appreciated. Also the authors would like to thank Dr. M. T. Davisson of the University of Illinois, Ken Keeler of Trumbull Asphalt, and Ed Minter and Henry Mathis of the Kentucky Department of Transportation for their suggestions concerning specifications/properties of bitumen for foundation piling.

## Note

The bridge and approach fills described in this article were constructed in early 1982. Instrumentation readings from strain gages installed on the two production piles and settlement gages installed on the fill confirm the estimates reported here. Additional detailed analysis currently is underway.

## Specification—Bitumen Coating for Steel Piles

*Description.* This work shall consist of furnishing and applying bituminous coating and primer to steel pile surfaces as required in the plans and as specified herein.

*Materials.*

A. Bituminous Coating. Canal liner bitumen (ASTM D 2521) shall be used for the bitumen coating and shall have a softening point of 88° C to 93° C (190° F to 200° F), a penetration of 56 to 61 at 25° C, and a ductility at 25° C in excess of 3.5 cm.

B. Primer. Primer shall conform to the requirements of AASHTO M 116.

*Construction Requirements.* All surfaces to be coated with bitumen shall be dry and thoroughly cleaned of dust and loose materials. No primer or bitumen shall be applied in wet weather nor when the temperature is below 18° C (65° F).

Application of the prime coat shall be with a brush or other approved means and in a manner to thoroughly coat the surface of the piling with a continuous film of primer. The purpose of the primer is to provide a suitable bond of the bitumen coating to the pile. The primer shall set thoroughly before the bitumen coating is applied.

The bitumen shall be heated to 149° C (300° F) and applied at a temperature between 93° C (200° F) and 149° C (300° F) by one or more mop coats or other approved means to apply an average coating depth of 10 mm (⅜ in). Whitewashing of the coating may be required during hot weather, as deemed necessary by the engineer, to prevent running and sagging of the asphalt coating prior to driving.

Bitumen-coated piles shall be stored immediately after the coating is applied for protection from sunlight and heat. Pile coatings shall not be exposed to damage or contamination during storage, hauling, or handling. Once the bitumen coating has been applied, the contractor will not be allowed to drag the piles on the ground or to use cable wraps around the pile during handling. Pad-eyes, or other suitable devices, shall be attached to the pile to be used for lifting and handling. If necessary, the contractor shall recoat the piles, at the contractor's expense, to comply with these requirements.

A nominal length of pile shall be left uncoated where field splices will be required. After completing the field splice, the splice area shall be brush or mop coated with at least one coat of bitumen.

*Method of Measurement.* Bitumen coating will be measured by the linear foot of coating in place on the pile surfaces. No separate payment will be made for the primer or coating of the splice area.

*Basis of Payment.* The accepted quantities of bitumen coating will be paid for at the contract unit price per linear foot, which price shall be full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in applying the bituminous coating and primer, as shown in the plans, and as specified in these specifications, and as directed by the engineer.

Payment will be made under:

| *Pay Item* | *Pay Unit* |
|---|---|
| Bitumen coating | Linear foot |

## Specification—Bitumen Coating for Concrete Piles

*Description.* This work shall consist of furnishing and applying bituminous coating and primer to prestressed concrete pile surfaces as required in the plans and as specified herein.

*Materials.*

A. Bituminous Coating. Bituminous coating shall be an asphalt type bitumen conforming to ASTM D 946, with a minimum penetration grade 50 at the time of pile driving. Bituminous coating shall be applied uniformly over an asphalt primer. Grade 40–50 or lower grades shall not be used.

B. Primer. Primer shall conform to the requirements of ASTM D 41.

*Construction Requirements.* All surfaces to be coated with bitumen shall be dry and thoroughly cleaned of dust and loose materials. No primer or bitumen shall be applied in wet weather nor when the temperature is below 18° C (65° F).

The primer shall be applied to the surfaces and allowed to dry completely before the bituminous coating is applied. Primer shall be applied uniformly at the quantity of 0.4 L/m$^2$ (1 gal/100 ft$^2$) of surface.

Bitumen shall be applied uniformly at a temperature of not less than 149° C (300° F) nor more than 177° C (350° F) and shall be applied either by mopping, brushing, or spraying at the project site. All holes or depressions in the concrete surface shall be completely filled with bitumen. The bituminous coating shall be applied to a minimum dry thickness of 3 mm (⅛ in), but in no case shall the quantity of application be less than 3.3 L/m$^2$ (8 gal/100 ft$^2$).

Bitumen-coated piles shall be stored before driving and protected from sunlight and heat. Pile coatings shall not be exposed to damage during storage, hauling, or handling. The contractor shall take appropriate measures to preserve and maintain the bitumen coating. At the time of the pile driving, the bitumen coating shall have a minimum dry thickness of 10 mm (⅛ in) and a minimum penetration value of 50. If necessary, the contractor shall recoat the piles, at the contractor's expense, to comply with these requirements.

*Method of Measurement.* Bitumen coating will be measured by the square yard of coating in place on concrete pile surfaces. No separate payment will be made for primer.

*Basis of Payment.* The accepted quantities of bitumen coating will be paid for at the contract unit price per square yard, which price shall be full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in applying the bituminous coating and primer, as shown in the plans, and as specified in these specifications, and as directed by the engineer.

Payment will be made under:

| *Pay Item* | *Pay Unit* |
|---|---|
| Bitumen coating | Square yard |

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**L. Radley Squier** is principal and founder of the firm L. R. Squier Associates, Inc. Dr. Squier has more than 20 years of diversified experience and is a staff consultant on difficult foundation problems.

# Recent Research Reports You Should Know About

**The following are brief descriptions of selected reports recently published by the Federal Highway Administration, Office of Engineering and Highway Operations Research and Development (R&D) and Office of Safety and Traffic Operations R&D. The reports are available from the address noted at the end of each description.**

## Traffic Evaluator System Users Manual, Vols. I–III, Report Nos. FHWA/RD-82/078-080

**by FHWA Office of Safety and Traffic Operations R&D, Systems Technology Division**

The Traffic Evaluator System (TES) is a large-scale highway data acquisition system developed by the Federal Highway Administration. The TES records time and event data on computer readable magnetic tape and can handle data rates and volumes that currently available commercial traffic survey equipment cannot accept. The TES with its associated cable network, sensor array, and portable power source is a highly flexible, cost effective tool for supporting the data collection and analysis needs of research and development efforts in the highway community.

Volume I, **Overview and Field Data Acquisition Guide,** focuses on the hardware/field equipment components of the TES. It presents in detail the relationships between components; planning requirements for a TES deployment; interpretative discussions of measures; procurement, preparation, and installation of hardware elements; and field operations and maintenance.

Volume II, **Data Processing Guide,** presents the computer programs needed to interpret and analyze the large quantities of data produced by the TES hardware. The program battery consists of a utility program to interpret raw data and an analysis program. Program functions, input requirements, and output options are discussed in detail.

Volume III, **Documentation of the TES Software,** provides several useful forms of documentation for the TES program battery. These include flow charted macrorelationships of the subroutine structure, "plain language" logical flow, details of dimensioned variables, and a complete FORTRAN source code listing of each program.

Limited copies of the reports are available from the Systems Technology Division, HSR-10, Federal Highway Administration, Washington, D.C. 20590.

## Social Impact Assessment: A Sourcebook for Highway Planners, Vols. I–VI, Report Nos. FHWA/RD-81/023-029


**by FHWA Office of Safety and Traffic Operations R&D, Urban Traffic Management Division**

These reports present a critical review of the technical literature and offer several detailed techniques that have potential application to social impact assessment (SIA).

Volume I is a users guide that indicates the need for performing SIA's and places such assessments in the context of the entire environmental impact and highway development processes. This volume also directs the user to the technical matters found in the other volumes.

The literature review, contained primarily in Volume II and its Addendum and Volume III, discusses social theory, identifies the kinds of social impacts, lists existing impact assessment techniques, and documents case studies. This information will provide a base for future research as well as direction to practitioners wishing to conduct social impact assessments for an actual project.

The specific techniques presented in Volumes IV through VI reveal both practical and innovative methods for dealing with the data collection and analysis aspects of performing SIA's in the planning and design of new or improved highway facilities. Volume IV explains the use of social data archives in impact assessment. Volume V discusses the use of associative group analysis for surveying public opinions. Volume VI offers guidelines for the development and administration of community surveys.

Limited copies of the set of reports are available from the Urban Traffic Management Division, HSR–40, Federal Highway Administration, Washington, D.C. 20590.

### Design of SEA Open Friction Courses, Report No. FHWA/RD–82/053

**by FHWA Office of Engineering and Highway Operations R&D, Materials Technology and Chemistry Division**

The combination of the anticipated shortage of asphalt cement and the projected abundance of sulfur has led to the investigation of the potential for substituting sulfur for asphalt cement in paving.

This research study was conducted to incorporate sulfur with asphalt to form sulfur-extended-asphalt (SEA) binders for use in open graded friction course (OGFC) mixtures. The experimental design variables included aggregate type, asphalt cement type, level of sulfur content in the binder, and method of preparing SEA binders.

Studies indicate that with minor modifications, the existing FHWA mix design procedure for conventional asphalt OGFC mixtures also may be used for preparing SEA-OGFC mixtures. In an analysis of variance study, it was found that minimal differences existed between binders produced by direct substitution method and emulsification. Test results also showed that the SEA-OGFC system exhibited improved structural, drainage, and freeze thaw properties over conventional OGFC systems.

Based on the test results obtained in this study, a proposed SEA-OGFC mix design procedure, patterned after the existing FHWA method, has been developed.

Limited copies of the report are available from the Materials Technology and Chemistry Division, HNR–40, Federal Highway Administration, Washington, D.C. 20590.

### Epoxy Thermoplastic Pavement Marking Material: Summary of Research Results and Revised Specification, Report No. FHWA/RD–81/144

**by FHWA Office of Engineering and Highway Operations R&D, Materials Technology and Chemistry Division**

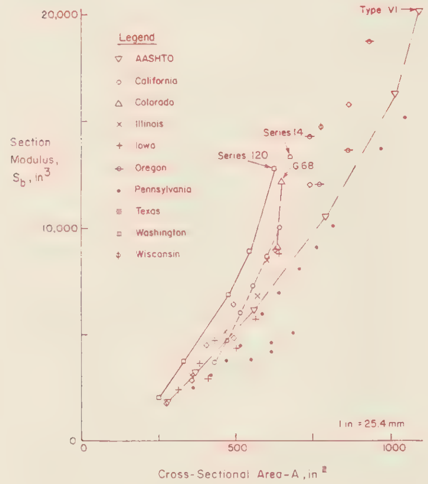
This report presents the results of an extensive laboratory program to establish a specification for an epoxy thermoplastic (ETP) striping material. The interim report for this study, Report No. FHWA/RD–80/069, developed laboratory test procedures to evaluate the properties of ETP, studied the effect of variations in ETP component ratios, and established an interim specification based on proprietary components for the ETP.

This report summarizes the results of an investigation of the effects of alternate pigments and resins on the ETP properties. The test methods and analytical procedures were modified and improved and were evaluated in an interlaboratory test program.

A revised specification was prepared which provides generic rather than proprietary description of the components of the white and yellow ETP's. Statistical procedures were used to establish confidence limits for the repeatability and the reproducibility of the various test procedures.

Limited copies of the report are available from the Materials Technology and Chemistry Division, HNR–40, Federal Highway Administration, Washington, D.C. 20590. The interim report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 82 135914).

### Optimized Sections for Major Prestressed Concrete Bridge Girders, Report No. FHWA/RD–82/005




**by FHWA Office of Engineering and Highway Operations R&D, Structures Division**

Some State highway agencies have developed their own standard sections for prestressed concrete solid form bridge girders while others use the American Association of State Highway and Transportation Officials (AASHTO) standard girder sections. This report analytically evaluates the various standard prestressed concrete bridge girder I-sections and T-sections in use in the United States to determine those suitable as national or regional standards for bridge spans in excess of 24.4 m (80 ft) using concrete with compressive strengths up to 48.3 MPa (7,000 psi).

Engineering information on current designs was collected from 11 State highway agencies and 3 prestressed concrete producers to assess the advantages and disadvantages of girder design concepts used in different areas. Fabrication, transportation, erection, and performance requirements of the different girder sections also were considered in this study. Dimensions and properties of all standard girder sections evaluated, including sections developed by State highway agencies, the AASHTO standard sections, and Bulb-T sections, are described in an appendix.

A practical consideration in determining optimum design sections is the web thickness requirement for adequate concrete cover and clear spacing between strands, assured consolidation of the concrete during manufacture in thin, deep members, and stability of slender members against buckling during transportation.

Single span bridge designs using various standard prestressed concrete I-section and T-section girders and cast-in-place concrete deck were analyzed on the basis of estimated inplace cost. Other variables included girder span, girder spacing, deck thickness, and concrete compressive strength.

The economic analyses were performed with computer program BRIDGE, which provided a cost index per unit roadway surface area for each bridge design. Output from the program also included midspan concrete stresses in top and bottom extreme fibers of the girder section at prestress transfer and under service load. In all cases considered, the bottom flange extreme fiber concrete stress at midspan under service load governed the design. Cost charts in the report provide a basis for comparing bridge designs with different standard girder sections in single spans of 24.4 m to 48.8 m (80 ft to 160 ft) as well as designs using the same girder sections with thicker webs.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 82 204033).

### A Probabilistic Approach to the Near Roadway Impact of Air Pollutants, Report No. FHWA/RD–82/100

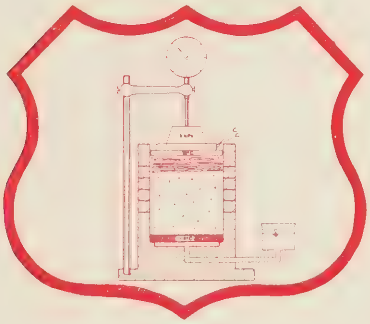
**by FHWA Office of Engineering and Highway Operations R&D, Construction, Maintenance, and Environmental Design Division**

This report discusses air quality standards of maximum time-average pollutant concentrations that may not be exceeded more than once per year. Primary consideration is given to impact locations near roadways. Methods for estimating the probability of violating air quality standards for inert gas contaminants such as carbon monoxide are presented with computer programs SIMCO1 and PROBCO1.

The predictions of nitrogen dioxide ($NO_2$) near roadways are also considered and a computer model, SIMNO 2, has been developed. Examples and comparisons are included to show that the methods and models are efficient and effective.

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

### Frost Susceptibility of Soil: Review of Index Tests, Report No. FHWA/RD-82/081



**by FHWA Office of Engineering and Highway Operations R&D, Construction, Maintenance, and Environmental Design Division**

Methods of determining the frost susceptibility of soils are identified and presented in this report. More than 100 criteria were found, the most common based on particle size characteristics. These particle size criteria frequently are augmented by grain size distribution, uniformity coefficients, and Atterberg limits. Information on permeability, mineralogy, and soil classification also has been used.

More complex methods requiring pore size distribution, moisture-tension, hydraulic-conductivity, heave-stress, and frost-heave tests have been proposed. However, none has proven to be the universal test for determining the frost susceptibility of soils.

Based on this survey, four methods were selected for further study—the U.S. Army Corps of Engineers Frost Susceptibility Classification System, the moisture-tension hydraulic-conductivity test, a new frost-heave test, and the California Bearing Ratio after-thaw test. Test results from these methods will be compared with field performance data from closely monitored test sections in New York and Massachusetts. Data will then be refined to produce an accurate method for assessing the frost susceptibility of soils.

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

### Material Properties to Minimize Distress in Zero-Maintenance Pavements, Vol. 1—Models, Report No. FHWA/RD-80/155, and Vol. 2—Parameter Study, Report No. FHWA/RD-80/156

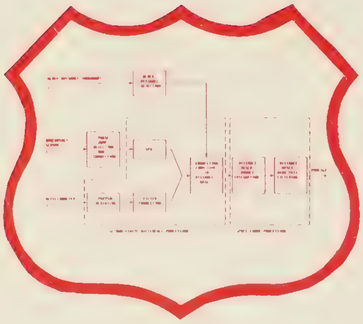
**by FHWA Office of Engineering and Highway Operations R&D, Pavement Division**

These reports provide results of a detailed study to identify distresses that cause significant loss of serviceability or maintenance in pavements; identify material properties that significantly influence the occurrence of distress; select the best theoretical or empirical models for predicting distress using material properties and other engineering parameters; and develop a detailed research plan for using the models selected to study the effects of the significant distresses and to optimize material properties for zero-maintenance pavements.

Volume 1 contains a set of definitions and examples to describe pavement behavior and the occurrence of distress; tabulations of distresses, material properties that affect specific distresses, and the material factors that affect specific material properties; discussions of the various predictive models available; and the results of limited sensitivity analyses using various distress models.

Volume 2 discusses the results of a parameter study with selected models using a range of material properties available in conventional and nonconventional pavement materials. The effects of varying these material properties on various distresses were evaluated and the tradeoffs required were discussed. The result from these studies was the development of a set of material properties that the models indicate were adequate to insure design of zero-maintenance flexible, rigid, and composite pavements.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 82 112905 and PB 82 112913).

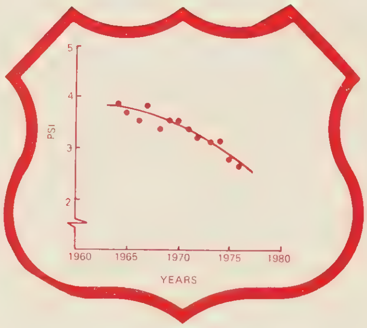


## Noncontact Road Profiling System, Vol. 1—Overview and Operating Manual, Report No. FHWA/RD-81/068, Vol. 2—Calibration and Maintenance Manual, Report No. FHWA/RD-81/069, Vol. 3—System Software, Report No. FHWA/RD-81/070, and Vol. 4—System Hardware, Report No. FHWA/RD-81/071

**by FHWA Office of Engineering and Highway Operations R&D, Pavement Division**

These reports discuss profile-measuring instruments. An accelerometer measures vertical vehicle motion and a noncontact sensor measures vertical pavement motion—either displacement or velocity—relative to the vehicle. Specifically, a method is developed to process the accelerometer and noncontact sensor signals to obtain a measured profile so that 0.15 m to 90 m (0.5 ft to 300 ft) wavelengths are recovered, there is no phase distortion, filtering and output are functions of distance rather than time, and the output is independent of data collection speed and direction. A hybrid processing technique involving a minimal amount of analog processing is used. The digital processing, which is now done offline, uses symmetric finite impulse response filters. The processing algorithms are described in detail, and a variety of results are presented.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 82 157355, PB 82 157363, PB 82 157371, and PB 82 157397).




## Relating Pavement Distress to Serviceability and Performance, Report No. FHWA/RD-80/098

**by FHWA Office of Engineering and Highway Operations R&D, Pavement Division**

This report examines available data on all kinds of pavement distress and on several forms of pavement performance to define their relationship in an understandable, predictable way. Regression analyses, Markov Processes, Bayesian Analysis, and utility theory are examined as analytical tools useful in relating distress to performance. Several specific performance models of limited applicability, generally applicable techniques, and recommended methods are reported.

This study shows that useful relationships can be obtained from existing data and are being used in several States. In each case, however, significant improvements in the distress-performance relationships are needed and can only be obtained through the conscientious observation, for approximately 10 years, of distress and performance of pavements under a variety of environmental and load conditions.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 81 227852).

## Computer Simulation of the Effect of Road Roughness on Tire-Pavement Forces for an Articulated Vehicle Performing Braking and Cornering Maneuvers, Report No. FHWA/RD-80/029

**by FHWA Office of Engineering and Highway Operations R&D, Pavement Division**

This report discusses an investigation of how road roughness affects the tire-pavement interface for an articulated vehicle. A modified version of the three dimensional vehicle simulation, TDVS3, was used to simulate braking and cornering maneuvers for an articulated vehicle on different road surfaces of varying severity.

The results of this investigation show that the effects of road roughness are greatest when the vehicle is performing maneuvers near its performance limit. For example, introducing road roughness to a severe cornering maneuver for which the vehicle has been stable, resulted in large enough increases in the vehicle's roll angle and lateral acceleration to cause the vehicle to become roll unstable. For a severe braking maneuver, introducing road roughness caused a 20 percent increase in the vehicle's stopping distance. These extreme effects were not manifested for less severe cornering or braking maneuvers that typically would be encountered in driving situations.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 81 238461).

### Feasibility Study for an Improved Acoustic Probe for Road Roughness Profiling, Report No. FHWA/RD-80/171

**by FHWA Office of Engineering and Highway Operations R&D, Pavement Division**

A feasibility study has been made of four alternative electro-acoustic sensor concepts for application to a noncontact acoustic road roughness profilometer. The concepts examined were a single-frequency phase detection system (already partially developed), a two-frequency phase detection system, an echo-ranging system, and an FM-ranging system. The study was primarily a paper investigation with some testing of critical circuits. Based on its simplicity, the echo-ranging concept merits further exploration. This report discusses some of the constraints and performance requirements on acoustic sensor systems and suggests a method of circumventing some environmental problems.

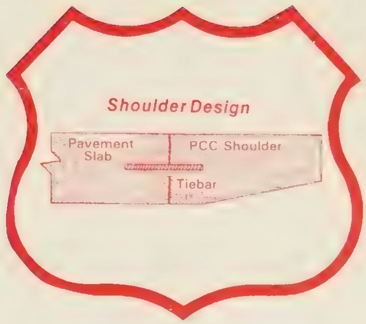
The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 81 236697).

### Structural Analysis and Design of PCC Shoulders, Report No. FHWA/RD-81/122

**by FHWA Office of Engineering and Highway Operations R&D, Pavement Division**

This report documents structural design of portland cement concrete (PCC) shoulders as related to subdrainage, shoulder structures, and maintenance of existing pavement systems or for new pavement construction.

Major factors that affect the behavior of PCC shoulders—encroaching moving trucks, parked trucks; foundation support, longitudinal joint




load transfer, shoulder slab thickness and tapering, width of shoulder, and traffic lane slab—are considered in the mechanistic design approach. The finite element structural analysis technique was used with a concrete fatigue damage model to sum damage for both moving encroaching trucks and for parked trucks. A relationship was established between the accumulated fatigue damage and slab cracking. Thus, the shoulder can be designed for an allowable amount of cracking that can vary depending on the performance level desired. Procedures for tying the PCC shoulder to the mainline PCC slab are recommended to provide adequate load transfer and to avoid joint spalling. Long term low maintenance performance of the PCC shoulder, with significant improvement in performance of the traffic lane, can be obtained for both new construction and rehabilitation if the shoulder is designed properly.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 82 255548).

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

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

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

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


## Highway Advisory Radio

**by Office of Implementation**

This slide-tape presentation will familiarize highway professionals with the concept of the highway advisory radio (HAR) system. HAR provides motorists with pertinent driving and travel information over their vehicle's standard AM radio receiver. HAR operates as a travel information station and is licensed under the Federal Communications Commission. Radio messages are transmitted on either 530 or 1610 kHz. The motorist's radio does not need to be modified or adapted.

Primary applications of HAR include traffic routing, parking information, and inclement weather advisories.

The slide-tape presentation explains the history, system components, design factors, transmission options, operational status, available documentation, and information sources.

The slide-tape presentation is available from FHWA Regional offices (see page 164); the National Highway Institute, HHI–20, Federal Highway Administration, Washington, D.C. 20590; and the Office of Implementation.

## Upgrading Safety Performance of Existing Bridge Rail Systems

**by Office of Implementation**

It is estimated that approximately 450,000 highway bridges may have substandard bridge rails. This 25-minute, color, sound motion picture presents economical techniques that can be used to retrofit the bridge rails. The movie shows the results of actual crash testing of retrofitted bridge rails and presents the design engineer with several retrofit systems available for immediate implementation.

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

## The Collection of Work Zone Accident Data, Report No. FHWA/RD–82/501, and Work Zone Accident Data Process, Report No. FHWA–IP–82–15

**by Office of Implementation**

Construction, maintenance, and utility activities often reduce the traffic carrying capacity of a roadway and increase hazards to motorists and workers. The dramatic increase in liability suits resulting from work zone accidents and the need for traffic control to be compatible with construction and maintenance activities have created the

need for a data collection and processing procedure whereby information from construction and maintenance sites is transmitted through regular communication channels to the State highway agency headquarters.

The development of this reporting procedure, including data collected from field trials in two States, is presented in Report No. FHWA/RD–82/501. A nine-State survey, which provided the basis for the procedure, is described in Report No. FHWA–IP–82–15.

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

### Traffic Controller Synchronizer—Field Test and Evaluation, Report No. FHWA–TS–82–220


**by Office of Implementation**

This report evaluates the results of the FHWA developed Traffic Controller Synchronizer (TCS). This unit is the predecessor to current commercial time based coordination units. The report will be of interest to traffic engineers concerned with low cost methods of improving traffic flow at signalized intersections.

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

### Time Based Coordination Unit—Specifications, Report No. FHWA–IP–82–20


**by Office of Implementation**

This report provides model specifications for a time based coordination unit, which is intended to provide coordinated operation of traffic signals without the need for interconnect cable. Coordination is achieved by maintaining very accurate time in each unit and developing the appropriate control signals on the basis of this time. Recommended minimum acceptance tests and typical application notes are included in the report.

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

### Isolated Traffic Actuated Program—Summary, Report No. NYDOT–IMP–81–1

**by Office of Implementation and New York State Department of Transportation**

This manual describes the capabilities of the enhanced Type 170 local intersection software (ITAP–170), including how it performs for isolated traffic actuated control. Two example applications—a six-phase diamond interchange operation and a five-phase operation for a five-leg intersection—are described. The ITAP is designed to function exclusively with the New York/California Traffic Signal Controller System. The software currently is being used to control several hundred intersections in New York and can accommodate most typical intersection control operations.

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

### Application of Traffic Simulation Models, Report No. FHWA-TS-82-207

**by Office of Implementation**

This report is a compilation of resource papers presented by State, city, and local transportation engineers at the Conference on Application of Traffic Simulation Models held in Williamsburg, Va., in June 1981. The report provides analytical procedures for evaluating major highway projects and selecting appropriate traffic control strategies (lane additions, turning pockets, and crossovers). Available traffic simulation and optimization models are discussed. The NETSIM (Traffic Network Simulation) model is highlighted with its uses in transportation management and traffic engineering, including quantifying vehicle related stops, delay, operating costs, fuel consumption emissions, and traffic costs.

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

### Equipment Management Symposium Proceedings Synopsis, Report No. FHWA-TS-82-217

**by Office of Implementation**

This report capsulizes presentations made at the Equipment Management Symposium in May 1981 in Coeur d'Alene, Idaho, on the development and use of equipment management systems. Major emphasis is placed on the presentations from four of the five States that contracted with FHWA to field test an equipment management system manual prepared through a pooled-fund study. Also included in the report are presentations made by representatives from organizations that have selected other courses of action.

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

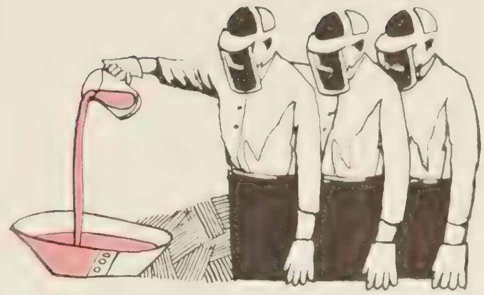
### Pavement Recycling: Summary of Two Conferences, Report No. FHWA-TS-82-224, and Proceedings of the National Seminar on PCC Pavement Recycling and Rehabilitation, Report No. FHWA-TS-82-208

**by Office of Implementation**

These reports summarize the results of national conferences on pavement recycling and rehabilitation. Guidance and recommendations to highway agencies are provided on the selection, design, administration, and evaluation of recycling projects. The summary report on two conferences includes both asphalt and concrete pavements.

Limited copies of both reports are available from the Office of Implementation.

# New Research in Progress



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

## FCP Category 1—Improved Highway Design and Operation for Safety

### FCP Project 1A: Traffic Engineering Improvements for Safety

**Title: Removal of Multiway Stop Signs Without Hazard. (FCP No. 31A1062)**
**Objective:** Develop and test a procedure to safely convert multiway Stop sign controlled intersections to two-way control. Review multiway Stop sign control conversions in 30 cities. Test a proposed procedure at three sites.
**Performing Organization:** AMAF Industries, Inc., Columbia, Md. 21044
**Expected Completion Date:** November 1983
**Estimated Cost:** $90,403 (FHWA Administrative Contract)

**Title: Limited Sight Distance Warning for Vertical Curves. (FCP No. 31A1954)**
**Objective:** Develop alternatives to the present limited sight distance sign for warning on crest vertical curves. Test the relative effectiveness of developed devices. Recommend warning device applications and uses. Compare the effectiveness of vertical curve warning devices with that of other traffic control devices.
**Performing Organization:** Ketron, Inc., Wayne, Pa. 19087
**Expected Completion Date:** September 1983
**Estimated Cost:** $121,958 (FHWA Administrative Contract)

**Title: Engineering Factors Affecting Traffic Signal Yellow Time. (FCP No. 31A2084)**
**Objective:** Develop a data collection manual for use by State and local traffic engineers to collect and analyze data relating to driver behavior at the onset of the yellow change interval.
**Performing Organization:** Texas A&M Research Foundation, College Station, Tex. 77843
**Expected Completion Date:** February 1984
**Estimated Cost:** $91,990 (FHWA Administrative Contract)

### FCP Project 1K: Accident Research and Countermeasure Effectiveness

**Title: Severity Indices for Roadside Features—A Pilot Study. (FCP No. 31K1152)**
**Objective:** Identify and catalog roadside features and identify and assess the adequacy of existing severity indices. Develop a study plan to obtain the severity indices of those features for which no information is available or which are currently under study.
**Performing Organization:** Texas A&M Research Foundation, College Station, Tex. 77843
**Expected Completion Date:** March 1984
**Estimated Cost:** $171,908 (FHWA Administrative Contract)

### FCP Project 1M: Rural Highway Operational Safety Improvements

**Title: Alleviation of Operational Problems on Two-Lane Highways. (FCP No. 31M2682)**
**Objective:** Investigate the traffic, safety, and cost effects of passing lanes, turnouts, shoulder use, two-way left turn lanes,and short four-lane road sections at 70 sites in 12 States. Collect traffic operational data at 30 sites. Define effects of site and traffic factors on performance by means of computer simulation.
**Performing Organization:** Midwest Research Institute, Kansas City, Mo. 64110
**Expected Completion Date:** November 1984
**Estimated Cost:** $383,095 (FHWA Administrative Contract)

**Title: Development of Optimum Edgeline Widths. (FCP No. 31M2723)**
**Objective:** Determine whether the use of 51 mm, 152 mm, or 203 mm (2 in, 6 in, or 8 in) edgelines as

compared to the 102 mm (4 in) edgeline is cost effective in reducing accidents. Design a controlled experiment to determine the optimum edgeline widths for various road and traffic characteristics.
**Performing Organization:** Wagner-McGee Associates, Alexandria, Va. 22304
**Expected Completion Date:** December 1983
**Estimated Cost:** $64,117 (FHWA Administrative Contract)

### FCP Project 1O: Railroad-Highway Grade Crossings

**Title: Cost Effectiveness Analysis of Using Railroad-Highway Crossing Active Warning Devices. (FCP No. 31O1194)**
**Objective:** Analyze and evaluate alternative active advance warning devices during a 1-year field demonstration. Develop recommendations for the most effective device.
**Performing Organization:** Goodell-Grivas, Inc., Southfield, Mich. 48075
**Expected Completion Date:** December 1984
**Estimated Cost:** $170,900 (FHWA Administrative Contract)

### FCP Project 1R: Achieving Safety Through Proper Speed Zoning and Control

**Title: Improved Techniques for Collecting Speed Data. (FCP No. 31R1014)**
**Objective:** Evaluate speed measurement, sampling, and analysis techniques in terms of accuracy and usefulness of data, observer variability, portability, labor requirements, and costs. Examine manual timing, radar, and automated methods. Investigate the source and magnitude of errors associated with speed data collection, including the effects of measurement methods on driver speed choice and day of week on which observations are made. Develop correction factors for known or measureable biases. Determine the variability of spot speeds over lengths of road and differences between free flow and all vehicle speed measures.
**Performing Organization:** Transportation Research Corporation, Haymarket, Va. 22069
**Expected Completion Date:** March 1984
**Estimated Cost:** $135,373 (FHWA Administrative Contract)

### FCP Project 1T: Advanced Vehicle Protection Systems

**Title: Evaluation of Design Analysis Procedures and Acceptance Criteria for Roadside Hardware. (FCP No. 31T5032)**
**Objective:** Develop recommendations for revision of National Cooperative Highway Research Program Report No. 230. Focus recommendations on filling in gaps, eliminating current assumptions, and expanding the current limits of highway safety appurtenance evaluation.
**Performing Organization:** Southwest Research Institute, San Antonio, Tex. 78284
**Expected Completion Date:** December 1985
**Estimated Cost:** $200,000 (FHWA Administrative Contract)

### FCP Project 1U: Safety Aspects of Increased Size and Weight of Heavy Vehicles

**Title: Field Test of the Grade Severity Rating System. (FCP No. 31U2063)**
**Objective:** Evaluate the best sign designs (based on simulator tests) in several States. Effectiveness measures will include brake temperature, truck speed, and safety.
**Performing Organization:** Transportation Research Corporation, Haymarket, Va. 22069
**Expected Completion Date:** August 1985
**Estimated Cost:** $268,553 (FHWA Administrative Contract)

**Title: Impact of Specific Geometric Features on Truck Operations and Safety at Interchanges. (FCP No. 31U2082)**
**Objective:** Determine what combinations of vehicle and roadway characteristics contribute to truck accidents at interchanges and develop possible countermeasures. Develop a user-oriented off-tracking model.
**Performing Organization:** University of Michigan, Ann Arbor, Mich. 48109
**Expected Completion Date:** September 1984
**Estimated Cost:** $228,674 (FHWA Administrative Contract)

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

**Title: Pavement Surface Conditions Measurement for Safety Improvements. (FCP No. 31W2044)**
**Objective:** Develop procedures for evaluating and quantifying pavement conditions at sites of frequent accidents or sites considered to be hazardous.
**Performing Organization:** Texas A&M Research Foundation, College Station, Tex. 77843
**Expected Completion Date:** March 1984
**Estimated Cost:** $216,782 (FHWA Administrative Contract)

FCP Project 1X: Highway Safety Program Effectiveness Evaluation

Title: Alternative Approaches to Accident Costs Concepts. (FCP No. 31X2152)
**Objective:** Identify the issues surrounding the differences in accident cost figures and develop study plans based on willingness to pay and capital cost approaches.
**Performing Organization:** Granville Corporation, Washington, D.C. 20005
**Expected Completion Date:** September 1983
**Estimated Cost:** $53,863 (FHWA Administrative Contract)

FCP Project 1Y: Traffic Management in Construction and Maintenance Zones

Title: Benefits and Safety Impact of Night Work Zone Activities. (FCP No. 31Y1164)
**Objective:** Develop guidelines for determining when highway maintenance should be done at night and the type and deployment of traffic control devices necessary.
**Performing Organization:** Virginia Highway and Transportation Research Council, Charlottesville, Va. 22903
**Expected Completion Date:** March 1984
**Estimated Cost:** $92,674 (FHWA Administrative Contract)

## FCP Category 2—Reduce Congestion and Improve Energy Efficiency

FCP Project 2K: Metropolitan Multimodal Traffic Management

Title: Integrated Traffic Data System. (FCP No. 32K2221)
**Objective:** Develop a microcomputer software package that will provide a computerized traffic data inventory file enabling the automation of the input process of various traffic simulation and optimization programs.
**Performing Organization:** Oak Ridge National Laboratory, Oak Ridge, Tenn. 37830
**Expected Completion Date:** September 1984
**Estimated Cost:** $150,000 (FHWA Administrative Contract)

FCP Project 2P: Improved Utilization of Available Freeway Lanes

Title: Changes Needed in Simulation Models to Test Freeway Bottleneck Solutions. (FCP No. 32P1023)
**Objective:** Assess the freeway simulation models INTRAS and FREFLO to determine the modifications required to simulate freeway bottlenecks and bottleneck solutions.
**Performing Organization:** JFT and Associates, Inc., Los Angeles, Calif. 90045
**Expected Completion Date:** November 1983
**Estimated Cost:** $150,600 (FHWA Administrative Contract)

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

Title: Laborsaving Methods for Improved Operation of Computer Controlled Traffic Signal Systems. (FCP No. 32Q1082)
**Objective:** Evaluate the feasibility of using automatically collected detector data to update Urban Traffic Control System signal timing plans.
**Performing Organization:** Kessmann and Associates, Houston, Tex. 77958
**Expected Completion Date:** November 1983
**Estimated Cost:** $199,961 (FHWA Administrative Contract)

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

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

Title: Highway Maintenance Impacts to Water Quality. (FCP No. 33E4102)
**Objective:** Identify maintenance practices that impact water quality and cost effective mitigation techniques. Develop guidelines for implementation of mitigation techniques and develop reference materials and guidelines for preparing environmental assessments.
**Performing Organization:** Dalton, Dalton, Newport, Inc., Cleveland, Ohio 44122
**Expected Completion Date:** June 1984
**Estimated Cost:** $106,850 (FHWA Administrative Contract)

## FCP Category 6—Improved Technology for Highway Construction

### FCP Project 6C: Use of Waste Materials for Highways

**Title: Mixture Design and Material Characterization of Cold Mixed Recycled Bituminous Pavements. (FCP No. 36C4374)**
**Objective:** Develop the optimum bituminous paving mixture design for representative cold mix recycled bituminous paving materials and evaluate their structural engineering properties. Determine the effect of material aging, environmental conditions, and traffic loads on the structural engineering and mixture properties. Establish the effectiveness of mixing and the rate of dispersion of the recycling agent to produce a homogeneous mixture. Evaluate the road performance and the structural engineering properties of field test specimens. Develop interim guidelines for the mixture design, evaluation of structural engineering properties, and pavement design.
**Performing Organization:** Resource International, Inc., Worthington, Ohio 43085
**Expected Completion Date:** September 1985
**Estimated Cost:** $145,000 (FHWA Administrative Contract)

### FCP Project 6E: Rigid Pavement Systems Design

**Title: Structural Design of Roadway Shoulders. (FCP No. 36E2062)**
**Objective:** Develop a practical and implementable procedure for the structural design of roadway shoulders.
**Performing Organization:** Resource International, Inc., Worthington, Ohio 43085
**Expected Completion Date:** September 1984
**Estimated Cost:** $52,700 (FHWA Administrative Contract)

### FCP Project 6K: Special Asphalt Concrete Mixtures

**Title: Investigation of Materials and Structural Properties of Asphalt-Rubber Paving Materials. (FCP No. 36K1034)**
**Objective:** Determine the relationships among binder, mixtures, and structural properties of asphalt-rubber systems and relate them to field performance. Develop appropriate specifications and recommend design and construction procedures for properly testing and handling the asphalt-rubber materials.
**Performing Organization:** Texas A&M Research Foundation, College Station, Tex. 77843
**Expected Completion Date:** September 1984
**Estimated Cost:** $176,631 (FHWA Administrative Contract)

### Federal Highway Administration Regional Offices:

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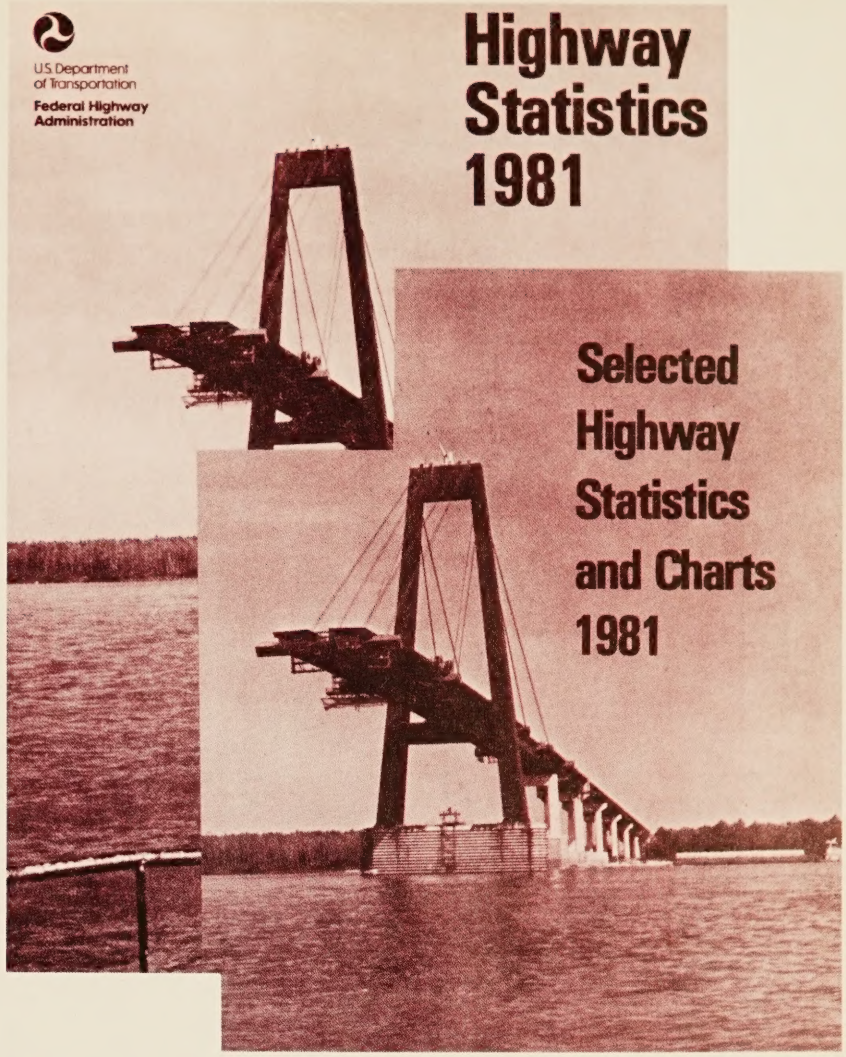


# New Publications

**Highway Statistics 1981,** a 176-page book, the 37th in the annual series, presents statistical and analytical tables of general interest on motor fuel, motor vehicles, driver licensing, highway user taxation, State and local highway financing, road and street mileage, Federal-aid for highways, and highway usage and performance. Also reported are 1980 highway finance data for municipalities, counties, townships, and other units of local government. A listing of the data is given in the table of contents, and a brief discussion is given in the text accompanying each section.

The publication may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–00256–6). It is also available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Report No. FHWA–HP–HS–81) in microfiche and paper copy.

The Highway Statistics series has been published annually since 1945, but the earlier editions, except 1975–1980, are now out of print. Much of the earlier data is summarized in **Highway Statistics, Summary to 1975**. These documents also may be purchased from GPO or NTIS.

**Selected Highway Statistics and Charts, 1981** is a 30-page compilation of statistical highlights and charts prepared as a convenient summary supplement to various tables published in **Highway Statistics 1981** and prior years. Historical trends, as well as 1982 estimates, are included. Copies may be obtained from the Highway Statistics Division, HHP–41, Federal Highway Administration, Washington, D.C. 20590.

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# in this issue

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