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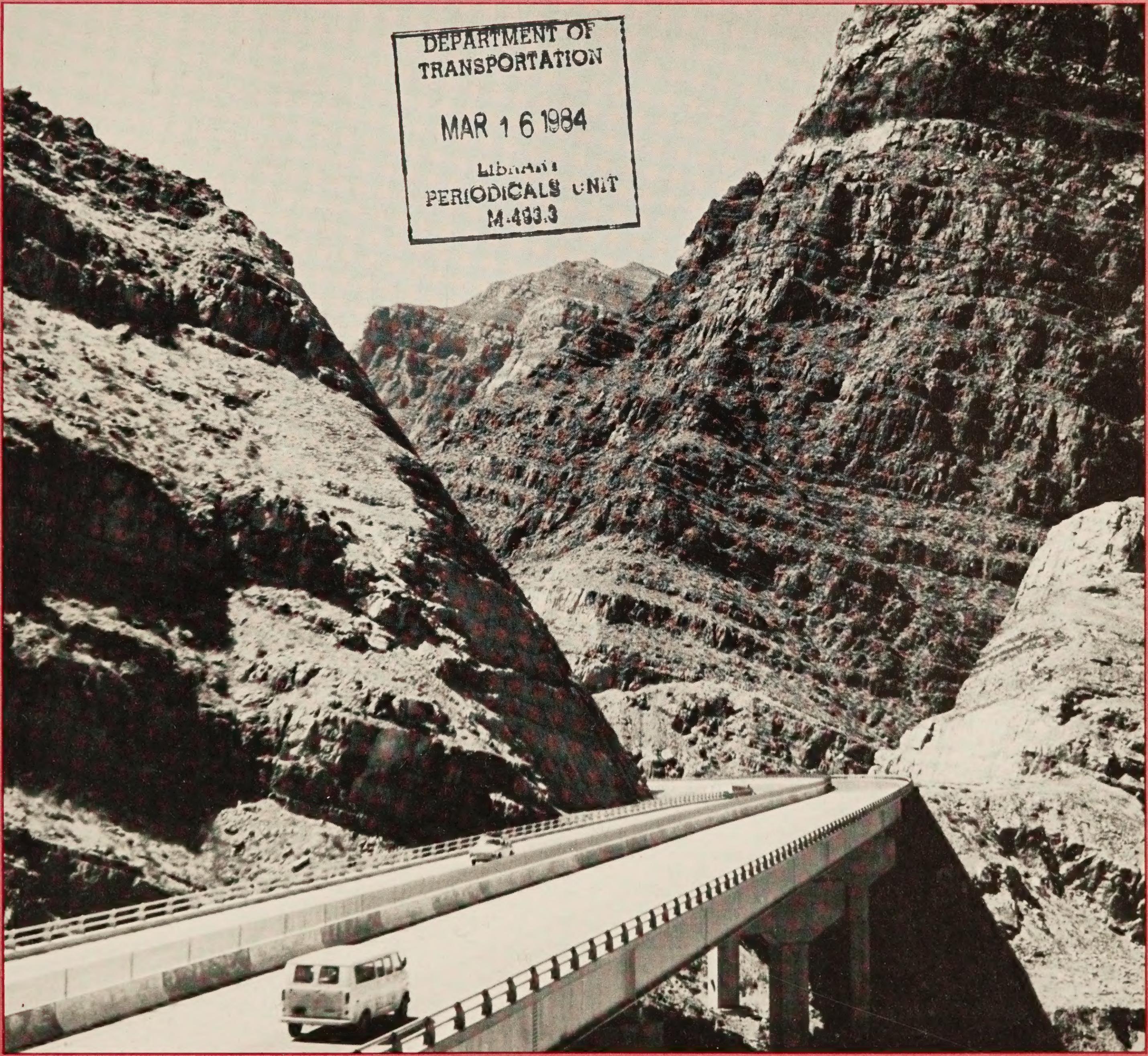

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COVER: Passage of interstate 15 through the Virgin River Canyon in Arizona.

IN THIS ISSUE

Articles

Departments

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Soda Springs, Idaho

Federal Highway Administration Research on Calcium Magnesium Acetate— An Alternative Deicer

by
Brian H. Chollar

Introduction

The importance of snow and ice control on highways and the advantages of a clear pavement cannot be overstated. In addition to greatly reducing highway accidents and fatalities (*1*)[1], keeping the roads clear has benefited the American economy by allowing the movement of people and goods.

Pavements in the snowbelt States usually are maintained during winter with salt (NaCl). Deicers such as salt melt snow by lowering the freezing point of water. The resulting solution flows naturally from the pavement. Freezing point depression is a colligative property; that is, a property that is dependent upon the number of particles in solution and not on the specific chemical properties of the elements that make up the compound. Therefore, an effective deicer should be highly soluble, ionize completely in solution, and be composed of atoms of low molecular weight. Salt meets most of these requirements. In addition, it is an abundant natural mineral—an economic incentive for its use. Though inexpensive and effective, salt can have its drawbacks including promoting corrosion of structural steel, spalling bridge decks, corroding vehicles, harming roadside vegetation, and degrading drinking water supplies. The chloride ion is a known accelerator of corrosion. The sodium ion, implicated in hypertension in human beings, alters the osmotic balance in living organisms.

The importance of maintaining the safety and traffic flow on our Nation's roadways during the winter months and the limitations being imposed on State transportation departments on the use of salt for deicing make it imperative that alternative deicing chemicals be developed. However, these alternatives must not have the adverse properties associated with the material they are replacing. Also, sufficient quantities of alternative deicers not as naturally abundant as salt must be able to be produced at a reasonable cost. The Federal Highway Administration (FHWA) and State highway and transportation departments are investigating the complete laboratory and field deicing, environmental, and economic effects of using calcium magnesium acetate (CMA), the most logical alternative to salt presently available.

FHWA-sponsored research to identify alternative deicers shows that CMA is potentially less polluting and corrosive than salt. CMA, which contains six ions or particles per molecule, contains neither sodium nor chloride ions

[1]Italic number in parentheses identifies reference on page 118.

and therefore would not have the same secondary effects as salt. It would be expected to have a different effect on metallic corrosion. The environmental impacts also would be expected to be different because sodium is not being added to the environment. Also, CMA has a higher solubility than salt, which means that it might be effective at lower temperatures. It is not naturally abundant and has to be produced from available starting materials.

Because of the potential deicing benefits of CMA, FHWA initiated a program in October 1982 for the complete evaluation of CMA as an alternative deicer. This article discusses the progress of this research over the past year and the plans for this program in the next 7 years.

Program Objectives

This program encompasses a complete evaluation of the physical, chemical, and environmental properties of CMA and the development of technology for producing, evaluating, and using CMA for snow and ice control. CMA, in both pure and commercial grades, will be evaluated for its environmental acceptability. CMA's deicing properties and corrosiveness to highway structures and vehicles will be measured. Conditions for the effective use of CMA will be determined, and an economic method for its production will be developed and demonstrated.

The program is organized into three emphasis areas:

Environmental acceptability of CMA—This task will measure and mitigate the environmental consequences of producing, storing, handling, and using CMA. The disposition, benefits, and deficiencies of CMA components in the receiving environment will be addressed, and the effect of CMA on water, soil, vegetation, and aquatic life will be analyzed.

Development of manufacturing technology—This task will develop an economical method for producing CMA from available raw materials. Procedures will be developed and optimized from literature and laboratory studies. Commercially produced material will be used for extensive field evaluations.

Technical and economic evaluation of CMA—This task will evaluate the technical merits of CMA and develop guidelines for its use. Technical laboratory evaluations of deicing ability and corrosiveness will be performed. Major field evaluations of deicing ability will be conducted. The performance data will be supplemented with economic considerations to develop a cost model to aid in selecting situations where CMA use is warranted and to determine proper techniques and guidelines for its use.

Current Studies in the CMA Program

Environmental studies of CMA

The California Department of Transportation used CMA synthesized from reagent chemicals to assess environmental consequences of using CMA. Potential effects of CMA on aquatic and terrestrial ecology, water quality, public health, ground water, and air are being evaluated. A literature search on water quality, ground water, public health, and air quality did not reveal any deleterious effects of CMA.

Fish bioassays (using fathead minnows and rainbow minnow fingerlings), zooplankton bioassays (using *Daphnia magna*), macro-invertebrate bioassays (using *Cheromus Plusmosis*), and phytoplankton bioassays (using *S. Capricornutum* and *A. Flos-aquae*) have been conducted. Results indicate that magnesium acetate is slightly more toxic and calcium acetate less toxic to fathead minnows and rainbow trout than is salt; an equimolar mixture of calcium and magnesium acetate is less toxic to rainbow trout than salt or either of the acetates tested separately. In addition, a continuously maintained 5,000 ppm level of an equimolar mixture of calcium and magnesium acetate slightly delays hatching of fingerlings but does not influence hatching success. Hatching difficulties observed in two cases (3 percent) at 5,000 ppm suggest that slightly higher levels could increase hatching mortality. A concentration this high in a natural environment would be unlikely and probably could occur only as a result of a deicer spill directly into the receiving stream.

CMA was not harmful to *Daphnia magna*. However, bacterial blooms occurred in CMA solutions at room temperature under laboratory conditions. *Daphnia magna* in test jars with bacterial blooms usually experienced complete mortality. The mortality may have been from oxygen depletion caused by the high oxygen demand of the bacterial bloom. This phenomenon made it necessary to establish two distinct LC_{50} (the concentration that causes death of 50 percent of the test subjects) values—one for CMA itself and one for the bacterial blooms that occur in CMA solutions. The results for macro-invertebrate bioassays are inconclusive.

The *S. Capricornutum* and *A. Flos-aquae* bioassays demonstrate that CMA, calcium acetate, and magnesium acetate are more deleterious to algae growth than salt. Statistically, the three highest treatment levels of CMA, beginning with the 0.0111 oz/gal (83.5 mg/L) treatment, and the two highest treatment levels for calcium acetate and magnesium acetate showed significantly less growth than the control or lower treatment levels for the unialgal bioassays.

The effects of CMA, calcium acetate, and magnesium acetate were decreased when diluted by stream water, but algae growth was still delayed (although these changes were statistically significant only at the highest treatment levels). A conservative estimate of the maximum concentration at which little effect would be observed from CMA, calcium acetate, or magnesium acetate would be 0.0067 oz/gal (50 mg/L) or less.

Laboratory studies were conducted to determine the effects of CMA and salt to terrestrial ecology using irrigation and foliar tests on 18 kinds of plants (fig. 1). The results of these tests show that for the plant irrigation treatments, nine plant species were most severely damaged by salt and one species by CMA. For eight other species, the difference in the degree of damage was not large enough to compare salt and CMA. Poor quality plants supplied with small root systems, dead foliage, and branches with few viable foliage buds masked the treatment-related damage. Comparisons between salt and CMA treatments were not possible with these poor quality plants. The organic planting mix for seven plant species strongly adsorbed the applied salt ions, interfering with the plant's ability to absorb the salt ions. The soil adsorption of salt ions buffered the treatment-related injury.

Spray treatments produced the greatest damage with salt for 17 of the 18 species tested (fig. 2). The 18th plant specie was of poor original quality; no comparison between salt and CMA was possible for this specie.

Deicer solution percolation studies also were conducted with seven kinds of soils. Soil treatment data were analyzed, and it is apparent that one normal CMA solution will pull significant amounts of iron and aluminum from the soils.

Manufacturing technology and field evaluation studies of CMA

A study to establish an economical procedure to produce CMA (Phase I) and then operate a pilot plant to produce 200 tons (181 Mg) of CMA for field deicing trials (Phase II) found that the most feasible way to produce CMA was to produce acetic acid and then react it with dolomitic lime (lime containing a high percentage of magnesium oxide). (*2, 3*) Many ways of producing acetic acid were investigated. The major criteria used in evaluating the total process to produce CMA were the purity of the reaction product, percent yield, ease of reaction, availability and cost of starting materials, and the conversion process per pound (kilogram) of CMA produced. A preferred process was identified and judged to have the maximum economic potential for commercial CMA production. The process entails the use of the anaerobic, thermophilic bacterium, *Clostridium thermoaceticum (C.t.),* to ferment biomass-derived sugars to acetic acid, followed by the reaction of the acetic acid with dolomitic lime.

Based on literature data for the wild strain of *C.t.,* this preferred process has three important advantages:

- The organism can make use of at least three sugars (xylose, fructose, and glucose) that can be derived from biomass by hydrolysis.
- The organism can produce exceptionally high yields of acetic acid—as much as 83 percent of theoretical yield.
- Acetic acid is essentially the only product, other than new cell mass, that is produced, thereby providing a route to an exceptionally uncontaminated CMA product.

Figure 1.—Application of CMA solutions to plant soils.

Figure 2.—Application of CMA solutions to plant foliage.

These high yields were verified in the laboratory when a glucose solution was converted to acetic acid under ideal laboratory conditions using this bacterial process.

An economic analysis of alternative processes for the large-scale production of CMA also was conducted. The analysis was based on a conservative estimate of the attainable performance of the preferred process with a reasonable amount of further development. The factors considered included feedstock and raw materials costs and availability, maintenance supplies and materials, labor, waste disposal, utilities, and fixed and capital-related costs. Three cases were considered in the economic analysis for a 1,000 ton/day (907 Mg/day) CMA production facility—conversion of whole shelled corn to CMA, conversion of glucose syrup to CMA, and production of CMA from acetic acid.

The cost of producing CMA depends greatly on feedstock availability and cost, with raw materials representing roughly one-half to three-quarters of the total revenue requirements for the three cases. Capital-related costs account for 13 to 27 percent of total revenue requirements. The results of this analysis indicated that the projected plant f.o.b. cost of CMA will be 18.8 cents/lb or $376/ton (41.4 cents/kg or $414/Mg) starting from corn and using the above bacterial process (table 1).

To conduct a field evaluation/demonstration of CMA, 200 tons (181 Mg) of CMA were produced by reacting purchased acetic acid with dolomitic lime (fig. 3). This pilot plant production was funded by FHWA and 24 States under a cooperative agreement. Under this agreement, each of the 24 participating States received 50 lb (22.7 kg) of CMA for evaluation, FHWA received 1 ton (0.907 Mg) of CMA for use in future studies, and the States of Michigan and Washington each received 98 tons (89 Mg) of CMA for field deicing evaluations under varying conditions during the winters of 1982/1983 and 1983/1984.

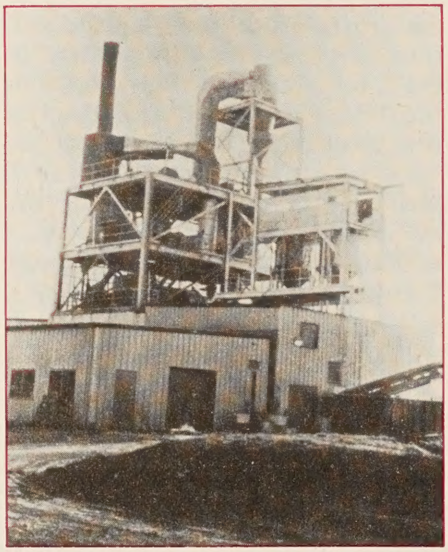

Figure 3.—The facility in Bowling Green, Kentucky, used to produce 200 tons (181 Mg) of CMA.

Table 1.—Production cost of CMA (dollars per pound of CMA)

Cost factors	Corn grain	Glucose syrup	Synthetic acetic acid
Materials and supplies	0.113	0.184	0.216
Labor	0.014	0.012	0.008
Purchased utilities and fuel	0.020	0.017	0.010
Fixed costs	0.009	0.009	0.006
Capital-related charges	0.060	0.047	0.033
Total	0.216	0.269	0.273
CMA	0.188	0.257	0.273
Feed byproduct	0.028	0.012	—

1 lb = 0.45 kg

On a February 1983 morning, Idaho applied CMA on a two-lane country road near Soda Springs. This test site had a 1½-in (38 mm) layer of packed snow when the test began, and the air temperature was 30°F (−1.1°C). Fifty pounds (22.7 kg) of CMA was applied on 250 ft (76.2 m) of the northbound lane on the site. As a control, 50 lb (22.7 kg) of salt was applied on the southbound lane of the site. The traffic density was very light—only four cars passed during the 3½-hour experiment. Within 2½ hours both deicing materials had achieved bare pavement. The salt performed slightly faster, but both obtained the same degree of bare pavement.

Michigan conducted field evaluations of CMA and salt on Interstate 96, a four-lane divided highway in Lansing with an average daily traffic volume of 24,000 vehicles. During the first snowstorm, which lasted 1½ days, CMA was applied on the eastbound lanes on seven passes at a rate of 400 to 800 lb/lane mile[2] (113 to 225 kg/lane km) during this 1½-day period. Salt also was applied on the westbound lanes on four passes at a rate of 250 lb/lane mile (70 kg/lane km) over the same 1½-day period. During this storm, most highways contained 1 to 2 in (25 to 51 mm) of hardpacked snow with ice underneath. Almost 8 in (203 mm) of snow fell, and the temperature was 17°F to 18°F (−8.3°C to −7.8°C) with 20 mph (32 km/h) winds during this period. Plowing was continuous. Results indicated that neither CMA nor salt was effective during the 1½ days of the storm. After the storm (third day), both the salt and CMA began to melt the snow effectively. Salt melted the snow at a higher rate than did CMA.

[2] According to theoretical considerations, the weight ratio of CMA to salt to obtain equal deicing capabilities is 1.7 to 1.0.

The second storm, one of short duration, deposited a light snow covering and caused bridge icing. During this storm, both CMA and salt were applied on four bridge deck overpasses on I–96 and on the roadways beneath to make sure bridge deck runoff did not refreeze. The temperature was 30°F (−1.1°C) during the storm. Results indicated that both CMA (applied at 400 lb/lane mile [113 kg/lane km]) and salt (applied at 200 lb/lane mile [56 kg/lane km]) were completely effective in removing and preventing ice and snow buildup on and under the overpasses within 2 hours of application.

The State of Washington did not have the adverse weather conditions necessary to field test the CMA material during the winter of 1982/1983.

A study to obtain a production-quality, mutant strain of bacterium based on *C.t.* has been initiated. The *C.t.* in the industrial process described above requires several changes to the wild strain. First, it must be able to tolerate as low a pH as possible to allow magnesium oxide (MgO) (from the dolomitic lime) to dissolve in the fermentation medium. Second, the bacterium should tolerate concentrations of magnesium, calcium, and acetate ions as large as possible in the medium to minimize subsequent evaporation requirements in producing the final CMA product. Last, the bacterium should maintain high acetic acid productivity, yield, and selectivity under the constraints of the two previous requirements.

The key objective of this study is the development of a mutant strain of *C.t.* that substantially fulfills these three requirements. In addition to this objective, accurate kinetic data for the wild strain of *C.t.* will be established for use as a baseline in evaluating subsequent mutant strains. Assuming that a useful mutant strain is developed, kinetic data will be gathered for feedstocks most likely to be used in an industrial fermentation process to produce CMA. CMA produced in a laboratory scale process then will be analyzed to determine the kind and quantity of active and inactive materials present.

Technical and economic evaluation studies of CMA

A study is underway to quantitatively determine the corrosion susceptibility of highway structural metals and other highway and bridge metals when exposed to CMA. These data will be compared with data from the corrosion process in deionized water and in calcium and sodium chloride solutions.

Another study has been initiated to optimize the deicing ability of CMA solutions and to define the components of solid CMA prepared by various laboratory and commercial production methods. The effects of varying the concentration and pH of CMA solutions and the calcium/magnesium ratio of CMA solids and solutions will be observed on the deicing properties of CMA. CMA's potential scaling of portland cement concrete also will be observed. The characterization of the CMA materials obtained from solution will be evaluated as will procedures for the purification of CMA and the compositional and morphological stability of CMA under storage conditions. The optimum deicing properties of CMA will be quantitatively compared with those of sodium and calcium chlorides.

Future Studies in the CMA Program

A new laboratory study planned for this year will evaluate mixtures of pure calcium and magnesium acetates and CMA produced in February 1983. This study will determine the corrosion and chemical effects of CMA on vehicle paint, metallic vehicle bodies, tires, chrome, and chassis metals. Chemical effects and corrosion rates will be quantified and compared with those rates and effects found for salt (sodium and calcium chlorides). The study also will determine the chemical effects of CMA on nonmetallic highway materials such as asphaltic and portland cement concretes, white and yellow traffic paint, and delineator and sign coatings. These effects also will be quantified and compared with effects found for salt (sodium and calcium chlorides).

In approximately 3 years a large-scale batch of CMA will be produced using the preferred process. This production will be based on the results of the research underway to obtain a production-amenable, mutant bacterial strain. The material from this process will be thoroughly evaluated in the field and laboratory, and a laboratory environmental survey will be conducted. The main emphasis of this study will be to determine the effects that the impurities in the CMA production sample, and the structure and composition of CMA made by this process, have on the environment.

Field deicing studies also will be conducted with this production batch of CMA. The deicing effects of CMA under all weather conditions, pavement types and conditions, and traffic and wheel loads will be determined. Varying application rates also will be tested. The deicing effectiveness of CMA mixtures with sand, cinders, and salt will be determined. The effects of applying CMA in solution and as a wetted solid also will be tested.

A thorough environmental study of CMA funded by the National Cooperative Highway Research Program is planned using the production batch at one or two field sites. Changes in plant growth, soil erosion, fish, algae, small animal habitation, runoff effects, and tree and plant foliage will be observed. Pavement effects also will be monitored during this test.

A handbook for the safe environmental use of CMA will be prepared. It will contain recommended procedures for application, amount, frequency of use, and form of CMA used under various weather, pavement, and climatological conditions and recommended geographical areas and environmental conditions under which the use of CMA will be economically advantageous over salt.

CMA will not replace salt as the predominant deicing chemical. However, in the foreseeable future CMA may be used instead of salt in certain areas (on bridge decks, in urban areas of high traffic volume, and in areas of possible contamination of water supplies by sodium) where the use of salt is prohibited because of corrosion or environmental reasons.

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(*1*) Robert Brenner and Jack Moshman, "Benefits and Costs in the Use of Salt to Deice Highways," *The Institute for Safety Analysis,* Rockville, Md., November 1976.

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(*3*) C. W. Marynowski, J. L. Jones, and E. C. Gunderson, "Production of Calcium Magnesium Acetate (CMA) for Field Trials," Report No. FHWA/RD–83/062, *Federal Highway Administration,* Washington, D.C., March 1984.

Brian H. Chollar is a research chemist in the Materials Technology and Chemistry Division, Office of Engineering and Highway Operations Research and Development, FHWA. He currently is the project manager of FCP Project 3C, "Calcium Magnesium Acetate (CMA) as an Alternate Deicer." He also manages contracts and conducts staff studies in several areas of materials research including delineation, alternative binder materials, chemistry of asphalts, and additives to asphalt. Before joining FHWA in 1974, Dr. Chollar was a research chemist at NCR Corporation involved with synthesis and analysis of organic materials including liquid crystals, organometallic compounds, and substituted porphyrins.

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

The Safety of Narrow Lanes for Traffic Control at a Construction Site

by
Willard J. Kemper, Harry S. Lum, and Samuel C. Tignor

Introduction

Safe and adequate traffic flow must be maintained during reconstruction of high-volume roadways. One traffic management technique used occasionally is establishing narrow lanes on the portion of the roadway left open to traffic. However, little information is available on the safety of narrow lanes in construction zones. A 1976 study of a construction project in England (*1*)[1] reported that narrow lanes were established on a two-lane road by converting the existing 12 ft (3.6 m) wide lanes and 10 ft (3.0 m) wide shoulders into two lanes in each direction—two 11 ft (3.4 m) wide lanes and two 9 ft (2.7 m) wide lanes with a median separation just over 3 ft (0.9 m) wide. The study concluded the following:

- Traffic delays would have quadrupled if the narrow lane scheme had not been used.
- Capacity and travel times through the narrow lanes were comparable with capacity and travel times at construction sites where standard lanes had been used.
- Although there were twice as many accidents during construction than before construction, this increase was less than that of other major construction projects, whether or not narrow lanes were used.

A 1979 study on the operation of 103 construction sites concluded that narrow lanes were particularly hazardous on high-speed roadways carrying nonlocal traffic. (*2*)

To gain further accident experience on the use of narrow lanes in construction zones, the Federal Highway Administration conducted a research study on the use of narrow lanes as part of a traffic control plan that was used for reconstructing and widening five bridge decks on the George Washington Memorial Parkway (GWMP) near Washington, D.C. The study was designed to examine how frequencies, kinds, costs, and reported causes of accidents differed between control and test (reconstruction) sections and how these factors differed between the 17-month period before reconstruction (December 1977 through April 1979) and the 17-month period during reconstruction (May 1979 through September 1980). This article discusses the results of the study.

Road and Traffic Conditions

The GWMP is a four-lane divided highway extending from the Capital Beltway (I–495) in Virginia southeast along the Potomac River toward Washington, D.C.'s central business district (fig.1). The posted speed limit is 50 mph (80 km/h). Commercial trucks are prohibited on the parkway.

[1] Italic numbers in parentheses identify references on page 124.

Because there was no convenient alternate route for traffic using the GWMP, the parkway was kept open to traffic during the bridge deck reconstruction. The test section was between the Route 123 interchange and Spout Run Parkway. Before reconstruction began, the average daily traffic (ADT) volume on this section of the parkway was approximately 60,000 vehicles. The control section, studied at the same time, was between the Route 123 and Interstate 495 interchanges. The control and test sections were 3.8 and 3.4 miles (6.1 and 5.5 km) long, respectively. There were no intersections or interchanges in the test section other than two scenic overlooks that were closed during the reconstruction.

The reconstruction was completed in three stages. During stage I, all traffic traveled on the 27 ft (8.2 m) wide outbound (from Washington, D.C.) side of the parkway, which was divided into three 9 ft (2.7 m) traffic lanes. Between 6 a.m. and 2 p.m., there were two inbound (toward Washington, D.C.) lanes and one outbound lane. Between 2 p.m. and 3 p.m. the center lane was closed, and traffic controls were changed for reverse traffic flow. Between 3 p.m. and 9 p.m. there was one inbound lane and two outbound lanes. Between 9 p.m. and 6 a.m. the center lane was closed to buffer the two opposing lanes.

During stage II, the inbound lane and the center lane were switched to the former inbound side of the parkway and the outbound lane remained on the other side.

In stage III, all three lanes were placed on the inbound side. During stages II and III, the same time schedule applied as during stage I. The posted speed limit was 35 mph (56 km/h) in the test section during all three stages (50 mph [80 km/h] in the control section).

During stage I the lanes were separated by 3 in (76 mm) polyethylene vertical tubes attached to an aluminum base with a slipjoint connection. The tubes would remain upright when hit at speeds less than 30 to 35 mph (48 to 56 km/h); at higher speeds, the tubes would break away and could be replaced undamaged or with only minor trimming of the tubes required. Vertical tubes sometimes were used during stages II and III to define 10 to 11 ft (3.0 to 3.4 m) traffic lanes. At other times, traffic cones were used in stages II and III. In all three reconstruction stages, traffic was confined to specific lanes by either vertical tubes or traffic cones (fig. 2).

The ADT within the test section was calculated using traffic counts taken during each of the three reconstruction stages; average weekday inbound

Figure 2.—Examples of vertical tubes used on the George Washington Memorial Parkway.

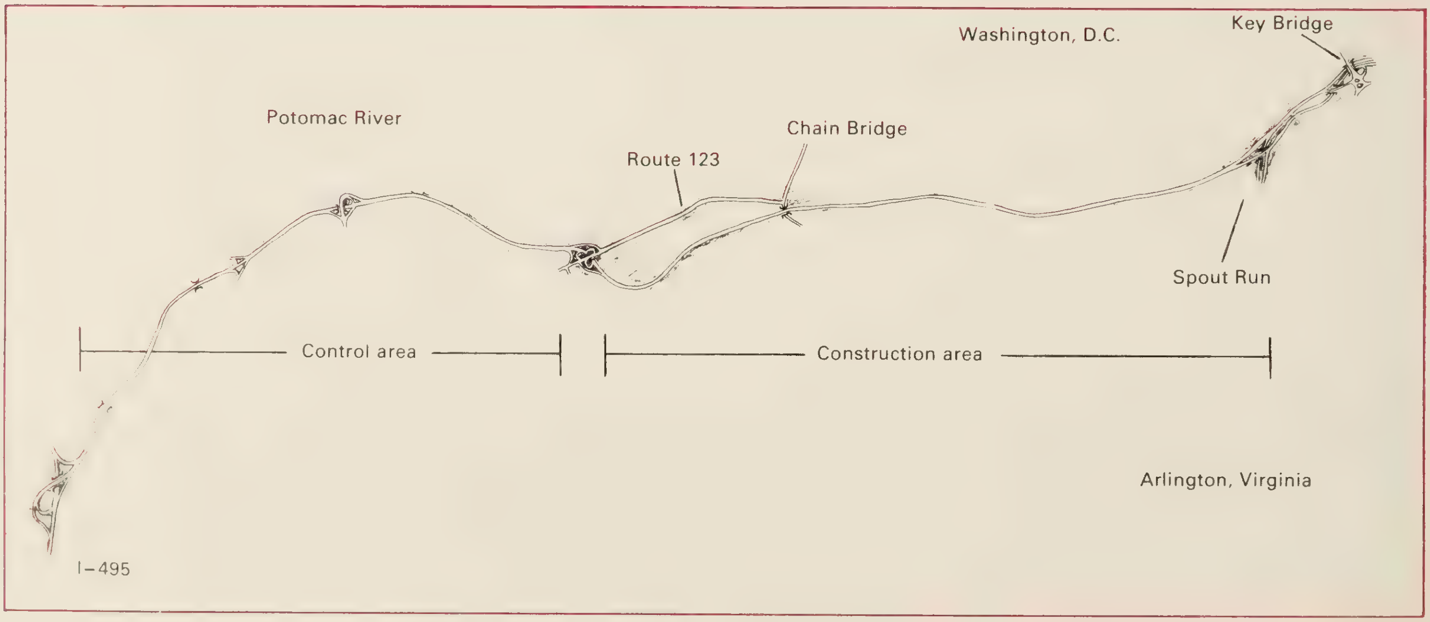

Figure 1.—George Washington Memorial Parkway.

and outbound counts were multiplied by five and average weekend counts were multiplied by two, the two counts added, and divided by seven. This ADT then was multiplied by 3.4 miles (5.5 km)—the length of the test section—and by the number of days in each stage to obtain the vehicle-miles (vehicle-kilometres) of travel. The distribution of vehicle-miles of travel (VMT) is shown in table 1. Because traffic counts were not taken in the control section during reconstruction, ADT and VMT had to be estimated and are shown only for comparison. The control section counts during reconstruction were estimated by multiplying the after-reconstruction counts in the control section by the same proportion of during-to-after-reconstruction counts observed in the test section.

Table 1.—Distribution of VMT

Parkway section	Stage	ADT[1]	VMT[1]
			Millions
Control	Preconstruction	46,500	91.1
	I	32,200	27.2
	II	42,900	23.5
	III	39,100	22.7
Test	Preconstruction	54,500	95.7
	I	40,300	30.4
	II	45,200	22.2
	III	44,800	23.3

[1] All ADT and VMT values in the control section are estimated.
1 mile = 1.6 km

Accident Frequency and Kind

Table 2 shows the accident frequency before and during reconstruction. There were three fatalities before reconstruction and one fatality during stage I. There were seven fewer injury and fatality accidents during the 17-month reconstruction period than during the 17-month period before reconstruction.

Table 2.—Accident frequency by accident severity before and during reconstruction

Stage	Severity: Property-damage-only accidents	Severity: Injury and fatality accidents	VMT
			Millions
Preconstruction	122	39	95.7 (55.8%)
I	64	16	30.4 (17.7%)
II	37	8	22.2 (12.9%)
III	33	8	23.3 (13.6%)
Total	256	71	171.6 (100.0%)

1 mile = 1.6 km

Table 3 shows accident severity rates per million vehicle-miles before and during reconstruction, and table 4 shows the kinds of accidents before and during reconstruction for the test and control sections. There were only five (2.1 percent) more accidents in the test section during reconstruction than before reconstruction. In the test section, collision-with-other-vehicle accidents increased 16.5 percent (from 115 to 134 accidents) during reconstruction and decreased 29 percent (from 41 to 29 accidents) in the control section during reconstruction. Collision-with-fixed-object accidents decreased 41 percent (from 39 to 23 accidents) in the test section during reconstruction and increased 69 percent (from 13 to 22 accidents) in the control section during reconstruction. These results suggest that traffic operations changed in both sections when the reconstruction began.

Table 3.—Accident severity rates per million vehicle-miles before and during reconstruction

Stage	Injury accidents	Fatality accidents	All accidents[1]
Preconstruction	0.38	0.03	1.68
I	0.49	0.03	2.63
II	0.36	—	2.03
III	0.34	—	1.76
Composite of stages I–III	0.43	0.01	2.18

[1] Includes property-damage-only accidents.
1 mile = 1.6 km

Table 4 shows a considerable difference in the kind and frequency of accidents occurring in the control and test sections, possibly because of heavier traffic volumes in the test section, merging traffic from Spout Run, and changes in traffic operations.

Table 4.—Accident frequency for test and control sections by kind of accident

Kind of accident	Time	Control section	Test section
Collision with other vehicle	Before reconstruction	41	115
	During reconstruction	29	134
Collision with fixed object	Before reconstruction	13	39
	During reconstruction	22	23
Other, noncollision	Before reconstruction	6	7
	During reconstruction	8	9
Total	Before reconstruction	60	161
	During reconstruction	59	166

Results of Analysis

An analysis, using the information statistic[2], of accident frequencies before and during reconstruction weighted by VMT found a statistically significant difference ($2I = 10.55$) at the 0.05 level in the accident frequencies before reconstruction and during the three stages of reconstruction. To determine which stage was causing the significant finding, an analysis was conducted between the preconstruction data and data from the three stages. The information statistic value found ($2I = 10.66$) indicates that the increase in the accident rate (also accident frequency) from 1.68 before reconstruction to 2.63 during stage I (table 3) is highly significant at nearly the 0.001 level. It should be noted that drivers were first exposed to the narrow lanes of the test section in stage I. There were no significant changes in accident frequency weighted by VMT in stages II and III. Further analysis found no statistically significant evidence of a change in the accident frequency among the three reconstruction stages.

Table 5 shows accident frequency before and during reconstruction for peak and offpeak periods. Again, a statistically significant difference ($2I = 22.45$ vs. $\chi^2_{.05,3} = 7.82$) in accident frequencies, when weighted by VMT, was found before and during reconstruction for the peak and offpeak periods. Also, the peak and offpeak accident frequencies, when weighted by VMT, were statistically different ($2I = 19.74$ vs. $\chi^2_{.05,1} = 3.84$). This difference is attributed to an increase in the offpeak accidents, possibly caused by an increase in drivers unfamiliar with the traffic pattern changes.

Reported Causes and Kinds of Accidents

Table 6 shows the reported cause of accidents in the test and control sections before and during reconstruction. The "following too close" and "not giving

[2] Because of its greater utility, the information statistic 2I, analogous to the Chi-square test statistic χ^2, was used in this analysis. *(3)*

Table 5.—Accident frequency before and during reconstruction for peak and offpeak periods

Stage	Peak	VMT	Offpeak	VMT	Total peak and offpeak	VMT
		Millions		*Millions*		*Millions*
Preconstruction	47	45.4	77	50.3	124	95.7 (55.7%)
I	29	13.1	49	17.3	78	30.4 (17.8%)
II	10	12.0	34	10.2	44	22.2 (12.9%)
III	13	11.7	25	11.6	38	23.3 (13.6%)
Total	99	82.2 (47.9%)	185	89.4 (52.1%)	284	171.6 (100.0%)

1 mile = 1.6 km

Table 6.—Reported cause of accident

Reason cited for accident	Accidents: Test section, Before reconstruction		Test section, During reconstruction		Control section, Before reconstruction		Control section, During reconstruction	
	Number	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
Following too close	44	27.3	62	37.4	12	20.0	9	15.3
Not giving full time and attention	53	32.9	57	34.4	22	36.7	17	28.8
Changing lane without caution	20	12.4	17	10.2	3	5.0	4	6.8
Improper turn	0	0.0	1	0.6	1	1.7	0	0.0
Right-of-way violation	2	1.3	1	0.6	2	3.3	0	0.0
Reckless driving	25	15.5	13	7.8	8	13.3	11	18.6
Speeding	5	3.1	2	1.2	4	6.7	10	16.9
Mechanical failure	1	0.6	3	1.8	1	1.7	0	0.0
Weather	3	1.9	1	0.6	2	3.3	1	1.7
Other (barricade in roadway, metal pipe in roadway)	8	5.0	9	5.4	5	8.3	7	11.9
	161	100.0	166	100.0	60	100.0	59	100.0

full time and attention" categories were both higher during reconstruction than before reconstruction in the test section. In the control section the opposite was observed. Also, the percent of accidents attributed to "reckless driving" and the percent attributed to "speeding" were both smaller in the test section during reconstruction, and again the opposite was observed in the control section. These differences may have been caused by some drivers driving more erratically in the control section after having become frustrated from being detained in the reconstruction section.

Table 7 shows the number and percentage of accidents occurring in the test section. Accidents are classified according to the first harmful event. For example, one of the accidents involved a pedestrian who was hit after the vehicle had run off the roadway to avoid a rear end collision. This accident was classified as a "ran off the roadway to avoid rear end" accident rather than a "hit pedestrian" accident. "Rear end" accidents accounted for over 65 percent of all accidents occurring during reconstruction. However, in a study of 2,127 work zone accidents in Virginia, 34.5 percent of the accidents were rear end accidents (*4*), and in a study of 2,948 rural construction zone accidents in Ohio, 40.4 percent were rear end accidents. (*5*) Thus, the proportion of rear end accidents that occurred during reconstruction on the parkway was almost twice as high as reported in previous studies, possibly because traffic was confined to the lanes by vertical tubes or cones. This indicates that most drivers stayed in the lane even if it resulted in an accident rather than run through the vertical tubes or cones.

Accident Cost

Table 8 shows accident frequency for control and test sections by cost, and table 9 is an analysis of these data. Because the duration of each stage (including preconstruction) was different and the lengths of the control and test sections were different, the significance of the main effects of stage, section, and cost has little meaning. What is important is interpretation of the significant interactions. The interactions between stage and section and stage and cost are significant at the 0.05 level, indicating an observed difference in the distribution of accident frequencies when categorized by accident cost, stage, and section.

Table 7.—Kind of accident reported

Accident	Stage I		Stage II		Stage III		Overall	
	Number	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
Rear end	52	65.1	24	53.4	34	83.0	110	66.3
Ran off roadway to avoid rear end	3	3.7	1	2.2	2	4.9	6	3.6
Forced off roadway	1	1.2	2	4.4	0	0.0	3	1.8
Ran off roadway	2	2.5	1	2.2	1	2.4	4	2.4
Sideswipe same direction	12	15.1	5	11.1	3	7.3	20	12.1
Sideswipe opposite direction	1	1.2	1	2.2	0	0.0	2	1.2
Hit cones or barricades	4	5.0	9	20.1	0	0.0	13	7.8
Hit construction equipment	1	1.2	1	2.2	0	0.0	2	1.2
Hit debris on roadway	4	5.0	1	2.2	0	0.0	5	3.0
Hit pedestrian	0	0.0	0	0.0	1	2.4	1	0.6
	80	100.0	45	100.0	41	100.0	166	100.0

Table 8.—Accident frequency by cost

	Cost[1]																
	Before reconstruction				Stage I				Stage II				Stage III				Total
	A	B	C	Total	A	B	C	Total	A	B	C	Total	A	B	C	Total	
Control section	59	25	12	96	18	14	4	36	3	4	4	11	14	7	5	26	169
Test section	141	85	46	272	118	38	12	168	42	28	19	89	64	26	8	98	627
	200	110	58	368	136	52	16	204	45	32	23	100	78	33	13	124	796

[1]A = < $500
B = $500–$1,000
C = > $1,000

Table 9.—Analysis of accident frequencies by stage, section, and cost

Component	Information	Degree of freedom	$\chi^2_{.05}$
Main effects:			
Stage	207.67[1]	3	7.82
Section	280.42[1]	1	3.84
Cost	239.99[1]	2	5.99
Interactions:			
Stage x section	13.85[1]	3	7.82
Stage x cost	27.22[1]	6	12.59
Section x cost	0.43	2	5.99
Residual (stage x section x cost)	11.57	6	12.59
Total	781.15[1]	23	35.17

[1] Significant at $\alpha = 0.05$.

The average cost of accidents in the test section before reconstruction was $590. Accident costs were lower in this section in stage I ($410), probably because most accidents were minor, property-damage-only accidents resulting from lower speeds on the narrow 9 ft (2.7 m) lanes. The high average cost for stage II ($1,050) results from two accidents—one involving an expensive car and the other involving an expensive piece of construction equipment. The average cost of accidents in stage III was $500.

Summary

This study has expanded upon the limited literature available on the use of narrow lanes in construction zones. Narrow lanes significantly affect accident rates and frequency as well as the kind and cause of accidents.

The public's first exposure to the narrow 9 ft (2.7 m) wide lanes caused a statistically significant increase in the accident rate, from 1.68 accidents per million vehicle-miles before reconstruction to 2.63 accidents per million vehicle-miles in stage I. The narrow lanes apparently slowed traffic, resulting in fewer injury accidents even though there were more accidents during reconstruction. During stages II and III, when 10 and 11 ft (3.0 and 3.4 m) wide lanes were used, the accident rates were less than those in stage I or before reconstruction.

The kind of accident was influenced greatly by traffic being confined to specific lanes. Rear end accidents were almost twice as frequent as reported in the literature. Also, in the test section, collision-with-other-vehicle accidents increased 16.5 percent during reconstruction, and collision-with-fixed-object accidents decreased 41 percent.

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(*3*) S. Kullback, M. Kupperman, and H. H. Ku, "Tests for Contingency Tables and Markov Chains," *Technometrics,* vol. 4, No. 4, November 1962.

(*4*) B. T. Hargroves and M. R. Martin, "Vehicle Accidents in Highway Work Zones," *Virginia Highway and Transportation Research Council,* December 1980.

(*5*) Z. A. Nemeth and D. J. Migletz, "Accident Characteristics Before, During and After Upgrading Projects on Ohio's Rural Interstate System," Transportation Research Record 672, *Transportation Research Board,* Washington, D.C., 1978.

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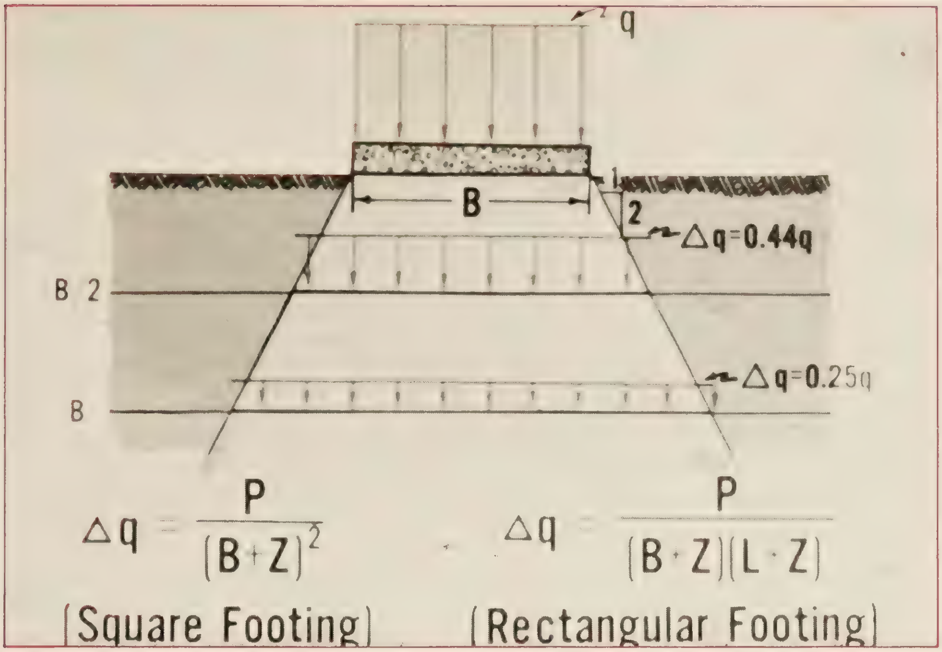
[3] Report with PB number is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.



Foundation Engineering Research: Part 1—Spread Footings

by
Albert F. DiMillio

This is the first of two articles on current efforts to improve highway bridge foundations. This article discusses spread footings and ground improvement techniques that can enhance the use of spread footings in lieu of piles. The foundation selection process is discussed briefly to illustrate how a designer can decide whether piles are necessary. Part 2, to be published in the June 1984 issue of *Public Roads,* will describe research efforts to improve pile design and installation methods.

Introduction

Foundation engineering is one of the oldest professions of mankind. The major decision in selecting a foundation system always has been whether to use shallow or deep foundations to support a superstructure. Foundation support is achieved by transferring the loads to the ground materials without incurring excessive settlements or distortions. Shallow foundations (spread footings or mats) normally are less expensive to construct than deep foundations (piles, drilled shafts, or caissons). However, surface soils usually are less capable of supporting heavy loads than deep soil deposits or bedrock. Deep foundations provide extra security against bearing capacity failure or settlement. A greater risk of unexpected engineering and contractual problems exists, however, during deep foundation installation. Engineering and cost analyses are necessary to determine the proper foundation system for major structures such as bridges, buildings, dams, and nuclear power plants.

The engineering analysis usually involves calculating stresses and strains in the foundation element and the surrounding soil. Other considerations are frost action, expansive soil pressures, scour, fluctuating water tables, environmental aspects, and the effect of nearby construction. Performance criteria also play a significant role in choosing the type, size, and location of the foundation elements for the structure.

The cost analysis determines which foundation system will provide the required support for the least cost, which is not necessarily the lowest initial cost. The size and cost of the superstructure affect the cost of the foundation system. The degree of risk and the consequences of failure also must be considered. It is estimated that during Fiscal Year 1981, the Federal Highway Administration (FHWA) and the State highway departments spent over $350 million on bridge foundations.[1]

The use of spread footings to support highway bridges in the United States varies widely between States and appears to be far below the optimum level. A recent survey by FHWA determined that most States use pile foundations to support the majority of their bridges.[2] This extensive use of piles to support U.S. highway bridges is contrasted by the extensive use of spread footings to support highway bridges in some foreign countries. For example, highway bridges in England seldom are founded on piles despite severe subsidence problems from

[1] S. N. Vanikar, "Manual on Design and Construction of Driven Pile Foundations," Federal Highway Administration, Washington, D.C., August 1983 (unpublished).

[2] "Foundation Engineering Management Reviews," Final Report, Office of Highway Operations, Federal Highway Administration, Washington, D.C., July 1982 (unpublished).

coal mining. Another contrast can be seen in the building industry where spread footings and mat foundations are used quite extensively even though building elements (for example, doors, windows, elevator shafts, and utilities) have less tolerance to settlement than bridges.

In the United States, many tons (megagrams) of pile materials and large sums of money are devoted to using pile foundations where spread footings might have been appropriate. The extensive use of piles in highway bridge foundations may be encouraged by the current American Association of State Highway and Transportation Officials bridge specifications, which state that "piling shall be considered when footings cannot, at reasonable expense, be founded on rock or other solid foundation material." (*1*)[3]

Some of the reasons piles are preferred over spread footings as a foundation for highway bridges include the following:

- The lack of well-documented performance evaluation data and the lack of rational tolerable movement criteria for bridges.
- Skepticism and uncertainties concerning the potential cost savings and the accuracy of settlement predictions for spread footings.
- Skepticism and uncertainties concerning the quality of fill or natural ground below the spread footings.

This article discusses each of these concerns and current FHWA research to examine their validity and increase the use of spread footings.

[3] Italic numbers in parentheses identify references on page 131.

Foundation Selection Process

The need for adequate bridge foundation support is well established, and a comprehensive and rational foundation selection process must be used to achieve adequate support.

Foundation economies occur mostly in the foundation selection process. If a spread footing can support a bridge abutment or pier adequately, minimal cost savings will be realized in the actual design of the footings. If a pile foundation is required, cost savings still are possible through a proper engineering analysis and selection of the type, size, and location of the piles.

The rational selection of a safe and economical foundation for bridge abutments and piers involves the relationships between subsurface conditions, structural loads, performance requirements, construction methods, and environmental effects. A systematic process of evaluating these factors is necessary to insure proper foundation selection. (*2*)

Subsurface conditions must be investigated thoroughly to choose and design the most economical foundation. The lack of proper soil and site information is a major reason pile foundations are used so frequently for bridges because an engineer will tend to select piles if the design data are incomplete. If proper soils information is available, the primary concern in soil support for structures is volume stability, strength, and durability. However, adequate support is highly variable and is more case-specific than site-specific. In the proper state, virtually any soil type, except highly organic materials, may be adequate for foundation support. Conversely, any soil type, in its natural state, may be inadequate for foundation support. Adequate soil support depends on the previously mentioned relationships between factors such as structural loads and performance requirements.[4]

Whether a given soil is adequate for foundation support and whether a given inadequate soil can or should be made adequate depend on the following factors that must be evaluated for each case:

- Soil type and present properties.
- Area and depth of soil treatment required.
- Local site conditions, including location of water table, presence of utilities, and obstructions of unknown location or size.
- Kind of structure to be supported.
- Level of improvement needed; for example, required bearing capacity, permissible settlements, and minimum relative density.
- Availability of foundation materials, skilled laborers, and equipment.
- Time available.
- Environmental factors, including waste disposal, ground water pollution, noise, and erosion.
- Cost relative to deep foundation alternatives.[5]

A group of piles is not more stable than a spread footing except when the piles can transfer the load through a weaker soil layer(s) to a firm, unyielding ground layer; or, in the case of excessive depth to a firm layer, the piles are driven deep enough into the weak layer(s) to develop practical refusal from frictional resistance. The load must be balanced against

[4] J. K. Mitchell, "Soil and Site Improvement Techniques to Enhance the Use of Shallow Foundations." Unpublished paper presented at FHWA's Research Review Conference, Atlanta, Ga., October 1977.

[5] Ibid.

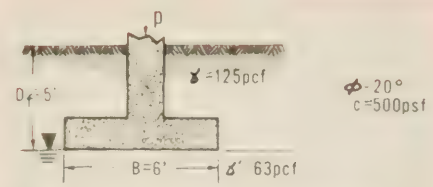
available soil support and performance requirements established by the bridge engineers for the particular structure being designed.

If the estimated costs of various foundation alternatives (determined during the design stage to be feasible) are within 15 percent of each other, alternate foundation systems might be included in contract bid documents.[6]

Spread Footing Design

The advent of reinforced concrete in the early 1900's and recent improvements in excavation technology have increased greatly the appeal of spread footings for bridge foundations. Modern soil mechanics and improved methods of site investigation and laboratory testing have improved the accuracy of settlement and bearing capacity predictions. Also, compaction control and improved grading procedures have minimized spread footings being founded on weak and/or compressible soils. Finally, special ground improvement techniques such as soil reinforcement, stone columns, and dynamic compaction have increased the attractiveness and applicability of spread footings.

Bearing capacity (fig. 1) and settlement (fig. 2) determine allowable foundation pressures. Usually, settlement is the controlling factor. In the design of spread footings, footing dimension proportions usually are based on 1 in (25 mm) of settlement and a safety factor of 2.5 or 3 with respect to bearing capacity failure. The bearing capacity is calculated from the estimated shear strengths of the supporting soils using the standard Terzaghi bearing capacity factors. (*3*) The magnitude and rate of settlement are computed from the compressibility properties of the foundation soils. (*4*)



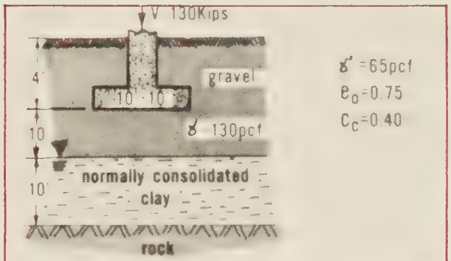
SOLUTION: Assuming a general shear condition from the Bearing Capacity Chart for $\phi = 20°$, $N_c = 17$; $N_q = 8$; $N_\gamma = 5$

For a square footing:

$$q_{ult} = 1.3cN_c + qN_q + 0.4\gamma' BN_\gamma$$
$$= (1.3)(0.5\text{ksf})(17) + (5')(.125\text{kcf})8 + 0.4(0.063\text{kcf})(6)(5)$$
$$= 11.0 + 5.0 + 0.75 = 16.75$$

$$\boxed{q_{all} = \frac{q_{ult}}{3} = \frac{16.75}{3} = 5.58 \approx 5.5\text{ksf}}$$

Figure 1.—The allowable bearing capacity of a square footing is determined using Terzaghi's bearing capacity equation and a safety factor of 3.



Overburden pressure at mid-height of clay deposit:

$$P_0 = (14' \cdot 130\text{pcf}) + (5' \cdot 65\text{pcf})$$
$$P_0 = 2145\text{psf}$$

Change in pressure at mid-height of clay deposit, using 2:1 method

$$\Delta P = \frac{V}{(10+15)^2} = \frac{130{,}000\#}{625\text{ft}^2} = 208\text{ psf}$$

$$\text{Settlement: } S = \frac{C_c H}{1+e_0}\log\frac{P_0+\Delta P}{P_0}$$

$$\boxed{S = \frac{(0.40)(10')}{1+0.75}\log\frac{2145+208}{2145} = 0.09' = 1.1\text{ins}}$$

Figure 2.—Determining the settlement of a spread footing.

Most highway agencies have a policy not to use spread footings on cohesive soils because of the concern for bearing capacity failure and/or excessive settlement. This is a conservative approach because some cohesive soil deposits (especially overconsolidated clays) can support heavy bridge loads without distress resulting. Although bearing capacity on sands is not a problem, many highway agencies do not use spread footings on cohesionless soils because of the concern for excessive settlement.

This also is a conservative policy because a spread footing on sand usually will provide satisfactory support because consolidation of sands usually is minimal and occurs rapidly. Most of the settlement occurs before the sensitive superstructure elements are erected.

The use of spread footings on compacted fill also is infrequent. Although a properly compacted fill often is stronger and more stable than natural ground and easily able to support a spread footing, designers often use spread footings on in situ soils and avoid their use on prepared fills. Large settlements from fills usually can be traced to older, nonuniform fills that may have been constructed of poor soils or uncompacted waste materials dumped on unprepared natural ground surfaces. However, the use of random, uncontrolled fill in modern highway construction, especially near bridges, is readily avoidable. The Connecticut and Washington State transportation departments frequently support bridge abutments on spread footings in compacted fill.

Preventing collapse of the spread footing is the main concern of the foundation designer; however, such failures are virtually nonexistent for highway bridges because of high safety factors. Failure also can be caused by excessive, long term settlement. Although collapse does not always occur, excessive settlement can severely crack the abutments and piers, or it can overstress key superstructure elements such as girders and deck slabs. This kind of failure results from design error or improper construction rather than from an inherent feature of spread footings. All too often, though, a failure caused by collapse or excessive settlement unduly discourages highway engineers from further use of spread footings.

[6] S. N. Vanikar, "Manual on Design and Construction of Driven Pile Foundations," Federal Highway Administration, Washington, D.C., August 1983 (unpublished).

A foundation system must be functional as well as safe. There is a wide degree of engineering performance between an unyielding support system and one that fails. Persistent maintenance problems and failures of noncritical elements (such as parapet walls and joints) are expensive to correct and should be avoided if peculiar to certain systems, situations, or methodologies. To improve the design process, engineers should correlate functional distress (bumps, cracks, and misalinements) with system characteristics (abutment type, soil type, superstructure type, and amount and kind of movement) to determine where spread footings are and are not appropriate.

Ground Improvement

When a spread footing design is inappropriate or the soils in their natural state are not able to provide adequate support, it is common to use piles or drilled shafts. However, this can be costly because sometimes it is more economical to improve the soils rather than use piles or remove and replace the unsuitable soils. Ground improvement methods—such as vibroflotation, Terraprobe, stone columns, Reinforced Earth, and dynamic compaction—generally fall in the following categories:

- Compaction by vibration.
- Compaction by displacement and vibration.
- Grouting and injection.
- Precompression.
- Reinforcement.

Many ground improvement methods originating in Asia and Europe are gaining widespread acceptance in the United States. Some methods have been used in the highway industry after successful use in the building industry. However, widespread acceptance of some methods has been hampered by poor quality assurance methods and a distorted economic picture. Each method has limitations, and some of the methods suffer from unrealistic design criteria. Other problems include the following:

- Unfamiliarity with the methods.
- Inequality of evaluation of the results obtained from the methods.
- Few specialty contractors available.
- Lack of technical education.

Many new ground improvement methods currently being examined are quicker, less expensive, more durable, and less disruptive to traffic operations than standard methods. For example, reinforced soil structures (fig. 3) reduce total costs by using earth fill instead of concrete, by reducing site preparation and installation time, by increasing tolerance to differential settlements, and by providing low-maintenance structures.

Current Research to Improve Shallow Foundation Design

Until recently, a rational and comprehensive evaluation of tolerable movements of highway bridges was not available, and designs for "zero" settlement were, and still are, common in the highway community.

The following are findings from a recent FHWA study to develop rational criteria for tolerable movements of highway bridges (*5*):

1. Depending on kind of spans, length and stiffness of spans, and the kind of construction material, many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without being overstressed, sustaining serious structural damage, or suffering impaired riding quality. In particular, a longitudinal angular distortion (differential settlement/span length) of 0.004 would most likely be tolerable for continuous bridges of both steel and concrete; an angular distortion of 0.005 would be a more suitable limit for simply supported bridges.

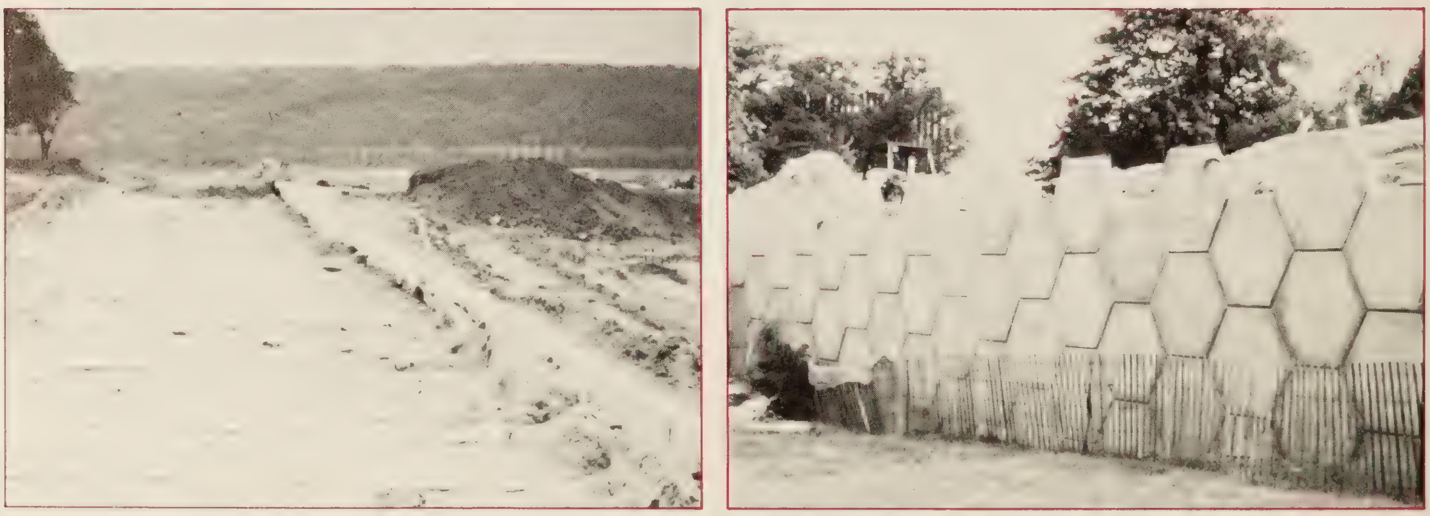


Figure 3.—(a) Metal reinforcing elements being installed in a Reinforced Earth wall. *(b) Nearly completed front facing of Reinforced Earth wall.*

2. An analytical evaluation of the effects of support settlements and dynamic vibrations on continuous steel bridges shows that the tolerance of any given bridge to such movements depends on a number of structural and geometric parameters of the system, such as flexural rigidity (EI), stiffness (I/ℓ), magnitude of differential settlement, number of spans, and span length.

3. For continuous steel bridges, differential settlements of 1 in (25 mm) or more would be intolerable for span lengths up to 50 ft (15.2 m) because of the significant increase in stresses caused by these settlements. However, for span lengths between 100 and 200 ft (30.5 and 61.0 m), the stress increases caused by differential settlements up to 3 in (76 mm) were quite modest, and for spans longer than 200 ft (61.0 m), the stress increases caused by 3 in (76 mm) differential settlements were negligible.

4. A basic design procedure was developed that permits a systems approach for designing the superstructure and the foundation system. This design procedure incorporates the tolerable movement guidelines that are based on strength and serviceability criteria which, in turn, are based on limiting longitudinal angular distortion, horizontal movements of abutments, and deck cracking.

5. Design aids such as a mathematical model, tables, and design charts also were developed to assist foundation engineers in determining potential behavior of continuous steel bridges subjected to differential vertical settlement.

To increase the number of documented case studies of spread footing performance, FHWA staff, in cooperation with the Washington State Department of Transportation, evaluated the performance of numerous highway bridge abutments supported by spread footings on compacted fill. During this review, the structural condition of 148 highway bridges throughout Washington State was visually inspected. The approach pavements and other bridge appurtenances also were inspected for damage or distress that could be attributed to the use of spread footings on compacted fill.

Based on this review and detailed investigations of the foundation movement of 28 selected bridges, it was concluded that spread footings can provide a satisfactory alternative to piles, especially when high embankments of good quality borrow materials are constructed over satisfactory foundation soils. None of the bridges investigated in Washington displayed any safety problems or serious functional distress; all bridges were in good condition.

In addition to the performance evaluation, cost-effectiveness analyses and tolerable movement correlation studies further substantiated the feasibility of using spread footings in lieu of expensive, deep foundation systems. Spread footings cost 50 to 65 percent less than pile foundations and foundation movement studies showed that the 28 bridges inspected easily tolerated differential settlements of 1 to 3 in (25 to 76 mm) without serious distress. (*6*)

FHWA recently initiated a 3-year comprehensive review of spread footing design and performance. The study will evaluate through the use of instrumentation the performance of 10 bridges supported on spread footings as well as review and evaluate current methods to calculate pressure distribution and settlement of spread footings on cohesionless soils (fig. 4). The methods, generally either empirical or based on elastic theory, vary in complexity and the treatment of ground water level and footing size. This study will identify the best method available, and parallel research will refine and improve the selected method.

Figure 4.—(a) Tiltmeter used to monitor abutment tilt from foundation settlement. (b) Optical settlement monitoring of bridge abutment wall supported by spread footing.

A study on stone columns as a foundation support for spread footings also was completed recently (fig. 5). (7) A study on dynamic compaction was initiated to improve guidelines for using this technique to stabilize soil masses for increased support of spread footings and roadway embankments (fig. 6).

Current research on ground improvement techniques that will significantly impact shallow foundation engineering includes a National Cooperative Highway Research Program (NCHRP) study on ground reinforcement and an FHWA study on prefabricated vertical drains. The NCHRP study will provide a comprehensive summary and evaluation of available

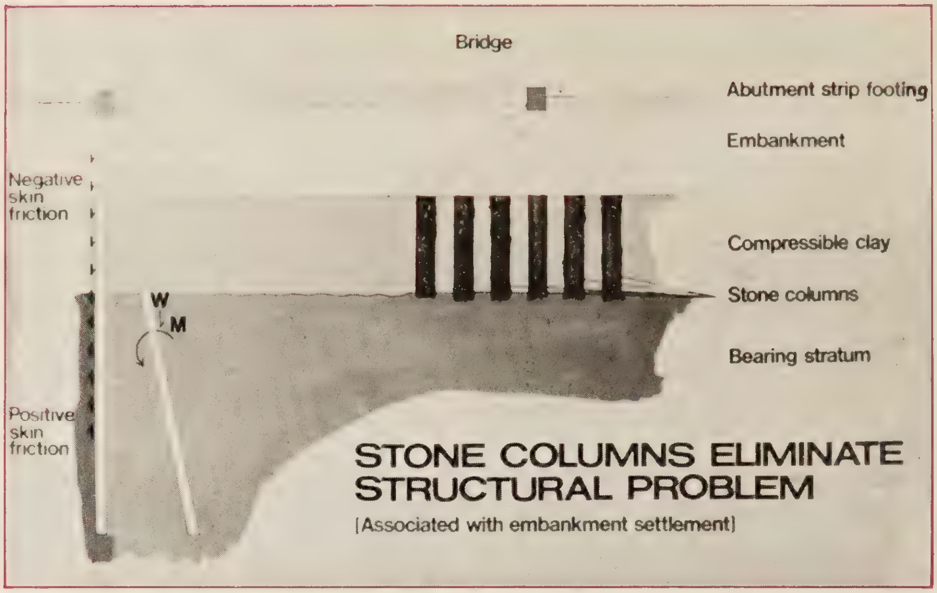
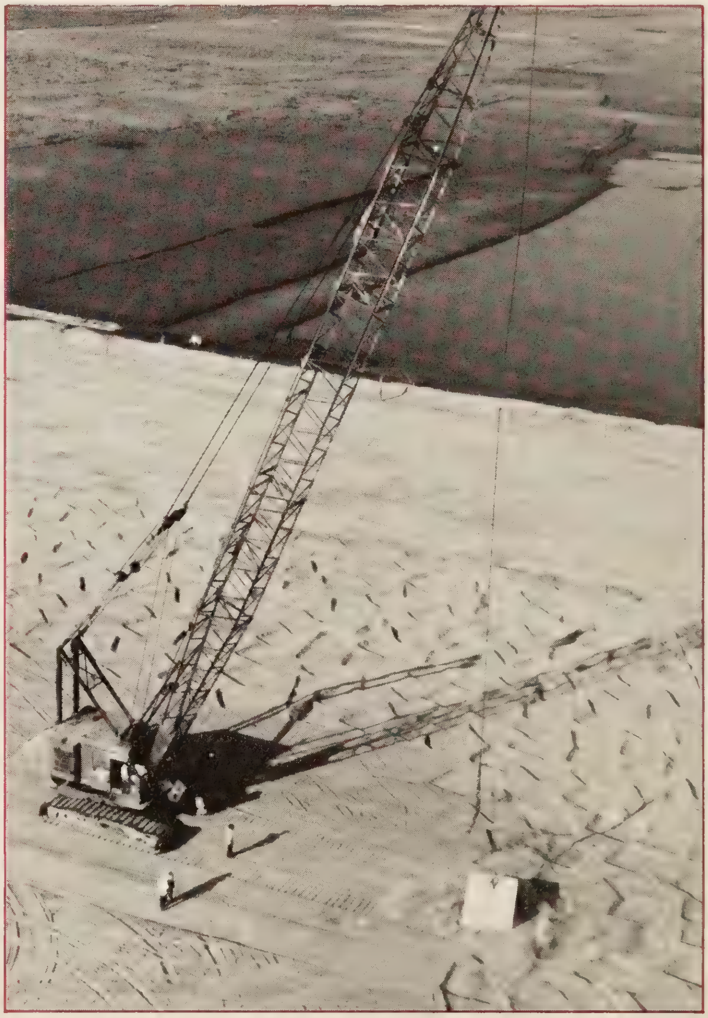


Figure 5.—Stone columns can eliminate structural problems.

Figure 6.—(a) Second pass of heavy tamping weight being dropped by 100 ton (91 Mg) crane to dynamically compact loose foundation soils.



(b) Last tamping pass to iron the surface to complete the compaction process.

earth reinforcement techniques applicable to highway embankment and slope stability problems. The FHWA study on prefabricated drains is not directed at shallow foundations but should have a significant positive impact on spread footing usage.

A future research effort on the behavior of reinforced soil should provide improved guidelines for designing and constructing reinforced soil systems using the most promising techniques identified under the NCHRP study. This effort will involve laboratory investigations of soil and reinforcement parameters and field pullout resistance tests on full-scale structures to evaluate design parameters.

Recent advances from the highway research program should reduce the skepticism, uncertainty, and concern for spread footing performance. Although the lack of well-documented performance data has stimulated research to develop a larger statistical data base on the safety and reliability of spread footings, additional performance evaluation studies of both shallow and deep foundations are needed. Bridge engineers should monitor the behavior of foundations and superstructures and correlate the performance of both to evaluate and improve the accuracy of current analytical and predictive techniques.

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(*5*) L. K. Moulton, H. V. S. GangaRao, and G. T. Halvorsen, "Tolerable Movement Criteria for Highway Bridges, Vol. I, Interim Report," Report No. FHWA/RD–81/162, *Federal Highway Administration,* Washington, D.C., September 1982. PB No. 83 198804.

(*6*) A. F. DiMillio, "Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill," Report No. FHWA/RD–81/184, *Federal Highway Administration,* Washington, D.C., October 1982. PB No. 83 201822.

(*7*) R. D. Barksdale and R. C. Bachus, "Design and Construction of Stone Columns," Report No. FHWA/RD–83/026, *Federal Highway Administration,* Washington, D.C., December 1983.

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

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How Effective are Crash Cushions in Reducing Deaths and Injuries?[1]

by
Lindsay I. Griffin III

In the last 20 years crash cushions have become fairly common features on major, high-volume highways throughout the United States. Typically, these attenuation devices are located immediately in front of massive, nonforgiving structures such as exit gores and bridge piers. Full-scale laboratory crash tests indicate that these devices should be highly effective in reducing deaths and injuries. But the questions remain: How effective are crash cushions? By what percentage do they reduce death and injury?

Several previous studies have considered the real-world performance of crash cushions *(1–3)*[2], but none has provided statistical estimates of the benefits attributable to these devices. The analyses that follow provide these statistical estimates, based upon a procedure developed in 1968. *(4)*

Analytical Procedure

When vehicles leave the road and strike nonforgiving obstacles such as the ends of bridges and supporting structures at underpasses, the occupants of those vehicles often are killed or injured. The likelihood of death and injury in these accidents is affected by combinations of factors such as accident location (rural/urban), kind of highway (Interstate, U.S. and State highway, or farm-to-market road), and kind of vehicle (passenger car, pickup truck, commercial truck, or "other" vehicle predominantly motorcycles). Table 1, which is based upon 4 years of Texas accident data (1978–1981), confirms that death rates and injury rates in single-vehicle accidents involving ends of bridges and supporting structures at underpasses are highly dependent upon different combinations of location and kind of highway and vehicle. For example, rural-Interstate-pickup truck accidents involving collisions with ends of bridges and supporting structures at underpasses produce 0.121 deaths and 0.845 injuries per accident. On the other hand, urban-U.S. and State highway-passenger car accidents involving collisions with the same kinds of structures produce 0.069 deaths and 0.660 injuries per accident.

Table 2 provides data on 560 single-vehicle accidents involving crash cushions. These accidents occurred on the Texas, State-maintained highway system between 1978 and 1981. The data in this table are broken down into the same 24 combinations of location and kind of highway and vehicle shown in table 1. The "expected" deaths and injuries in

[1] This article is adapted from the July 1983 issue of the *Texas Transportation Researcher*, Vol. 19, No. 3.

[2] Italic numbers in parentheses identify references on page 134.

Table 1.—Accidents, deaths, and injuries associated with ends of bridges and supporting structures at underpasses

Location	Highway	Vehicle	Accidents	Deaths per accident	Injuries per accident
Rural	Interstate	Car	208	0.120	0.591
		Pickup	58	0.121	0.845
		Truck	66	0.303	0.439
		Other	9	0.111	0.889
	U.S. and State	Car	390	0.179	0.687
		Pickup	198	0.121	0.586
		Truck	82	0.146	0.622
		Other	12	0.083	0.500
	Farm-to-market	Car	176	0.153	0.653
		Pickup	100	0.190	0.660
		Truck	10	0.100	0.600
		Other	2	0.000	1.000
Urban	Interstate	Car	432	0.118	0.771
		Pickup	107	0.075	0.673
		Truck	52	0.058	0.500
		Other	12	0.000	0.500
	U.S. and State	Car	420	0.069	0.660
		Pickup	109	0.064	0.734
		Truck	45	0.022	0.467
		Other	9	0.111	0.778
	Farm-to-market	Car	41	0.049	0.512
		Pickup	20	0.050	0.400
		Truck	3	0.000	0.000
		Other	0	—	—
	Total		2,561		

Table 2.—Accidents, deaths, and injuries associated with crash cushions

Location	Highway	Vehicle	Accidents	Expected Deaths	Expected Injuries	Observed Deaths	Observed Injuries
Rural	Interstate	Car	10	1.20	5.91	0	3
		Pickup	4	0.48	3.38	0	2
		Truck	0	—	—	0	0
		Other	1	0.11	0.89	0	0
	U.S. and State	Car	15	2.69	10.31	0	7
		Pickup	4	0.48	2.34	1	2
		Truck	2	0.29	1.24	0	2
		Other	0	—	—	0	0
	Farm-to-market	Car	3	0.46	1.96	0	5
		Pickup	3	0.57	1.98	0	0
		Truck	1	0.10	0.60	0	0
		Other	0	—	—	0	0
Urban	Interstate	Car	258	30.46	198.88	5	128
		Pickup	79	5.91	53.16	3	40
		Truck	23	1.33	11.50	0	9
		Other	13	0.00	6.50	0	8
	U.S. and State	Car	92	6.35	60.68	2	55
		Pickup	29	1.86	21.28	1	9
		Truck	5	0.11	2.33	0	3
		Other	8	0.89	6.22	0	11
	Farm-to-market	Car	6	0.29	3.07	0	2
		Pickup	4	0.20	1.60	0	0
		Truck	0	—	—	0	0
		Other	0	—	—	0	0
	Total		560	53.78	393.83	12	286

this table were derived by multiplying the death and injury rates shown in table 1 by the number of accidents shown in table 2. For example, table 2 shows there were 10 rural-Interstate-passenger car accidents involving collisions with crash cushions. Had crash cushions not been installed at the locations where these 10 accidents occurred, these accidents would, no doubt, have involved collisions with massive, nonforgiving obstacles such as the ends of bridges or supporting structures at underpasses. Table 1 shows that rural-Interstate-passenger car accidents involving collisions with ends of bridges and supporting structures at underpasses produce 0.120 deaths and 0.591 injuries per accident. Applying these coefficients to the 10 accidents in table 2, it is expected that these accidents would have produced 1.20 deaths and 5.91 injuries had crash cushions not been used at the locations where the accidents occurred.

Results

Total expected deaths and injuries in the 560 accidents in table 2 are 53.78 and 393.83, respectively. But, only 12 deaths (78 percent fewer than expected) and 286 injuries (27 percent fewer than expected) occurred in these 560 accidents. In other words, on the basis of these analyses it can be concluded that crash cushions reduce fatalities by 78 percent, and injuries by 27 percent.

It was further determined by analyses identical to those discussed that crash cushions reduce serious (A-level) injuries by 67 percent, moderate (B-level) injuries by 8 percent, and minor (C-level) injuries by 12 percent.

These findings are in good agreement with a study conducted by the Federal Highway Administration in 1973. In that study, 68 crash cushion accidents were judged likely to have resulted in deaths or A-level injuries had crash cushions not been present. Yet, of the 68 accidents, only 5 resulted in deaths and 12 in A-level injuries. Thus, the estimated reduction in deaths and A-level injuries was 75 percent in that study. *(2)*

Finally, it should be recognized that when crash cushions are placed near the roadway, they constitute, in and of themselves, an additional obstacle in the roadway environment. Accordingly, the installation of crash cushions should be expected to increase accident rates somewhat, thereby reducing the savings in deaths and injuries cited above. The extent to which crash cushions may increase the accident rate is unknown; but, if good engineering practice is followed, it is thought that the increase will be small.

REFERENCES[3]

(1) Monroe C. White, Don L. Ivey, and T. J. Hirsch, "In-service Experience on Installations of Texas Modular Crash Cushions," *Texas Transportation Institute, Texas A&M University*, College Station, Tex., December 1969.

(2) John G. Viner and C. M. Boyer, "Accident Experience With Impact Attenuation Devices," Report No. FHWA–RD–73–71, *Federal Highway Administration*, Washington, D.C., April 1973. PB No. 224995.

(3) T. J. Hirsch, John F. Nixon, Dave Hustace, and E. L. Marquis, "Summary of Crash Cushion Experience in Texas—Four Hundred Collisions in Seven Years on 135 Installations," *Texas Transportation Institute, Texas A&M University*, College Station, Tex., November 1975.

(4) B. J. Campbell, "Seat Belts and Injury Reduction in 1967 North Carolina Automobile Accidents," *The University of North Carolina Safety Research Center*, Chapel Hill, N.C., December 1968.

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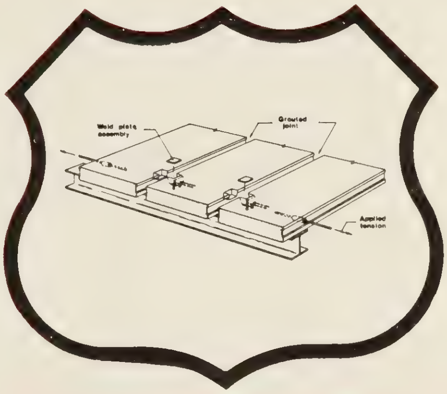
[3]Report with PB number is available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

Recent Research Reports You Should Know About

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

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



Connections for Modular Precast Concrete Bridge Decks, Report No. FHWA/RD–82/106

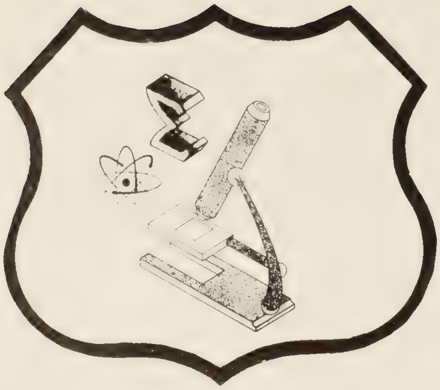
by Structures Division

This report presents the results of a study that examined and evaluated connections between bridge deck elements and between bridge decks and supporting girders. The first part of the report presents a comprehensive state-of-the-art summary of precast concrete use in bridge decks with an emphasis on connection methods and devices. Three kinds of decks were examined: Stay-in-place forms for cast-in-place decks, full depth precast decks on girders, and precast, usually prestressed, integral (multibeam) decks.

A variety of connection types and details are examined and evaluated including both deck-to-girder and deck unit-to-unit connections. A thorough study of grout materials and lateral tie methods is included.

The second part of the report examines a two-phase laboratory test of two different kinds of bridges and connections. Phase 1 was a pilot program that would identify the variables and suggest methods of simulating the variables involved in shear key and lateral restraint design. Two tests were made with different restraint systems and grout key materials. Phase 2 was a series of static and dynamic tests of connections from precast decks to prestressed concrete girders. Suggested design procedures and a design example are included.

The report may be purchased from NTIS.



Kiln Dust-Fly Ash Systems for Highway Bases and Subbases, Report No. FHWA/RD–82/167

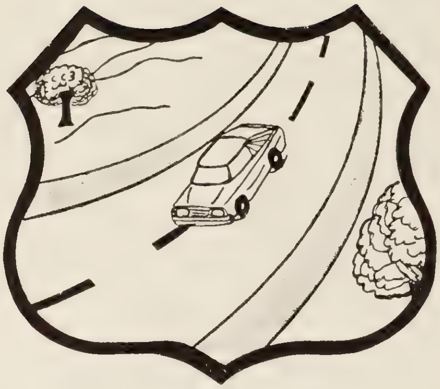
by Construction, Maintenance, and Environmental Design Division

This report presents the results of a laboratory study to evaluate the effectiveness of substituting kiln dust for hydrated lime in lime-fly ash aggregate road base systems.

Forty-five kiln dust samples, including 33 cement dusts and 12 lime dusts, were obtained in accordance with a standard sampling procedure. In addition, 18 fly ashes (including 5 Class C ash samples) and 6 aggregates were included in the sampling program.

The results of the study indicate the majority of the kiln dusts react with fly ash and aggregate to provide a base material with engineering properties such as strength, durability, and dimensional stability equal to those of conventional lime-fly ash mixtures. The report includes recommendations for selecting, testing, and designing kiln dust-fly ash aggregate mixtures for pavement bases and subbases. Because the results of the laboratory study demonstrate that kiln dusts have high potential for cost and energy savings, the report recommends that field trials be initiated as soon as possible to verify laboratory results.

The report may be purchased from NTIS.



Accident Surrogates for Use in Analyzing Highway Safety Hazards, Vol. I, Executive Summary, Vol. II, Technical Report, and Vol. III, Appendixes A–G, Report Nos. FHWA/RD–82/103–105

by Safety and Design Division

An accident surrogate measure is defined as a quantifiable observation that can be used in place of or as a supplement to accident records. These reports provide evidence that surrogate measures for accident experience can be identified. A procedure for developing and using accident surrogates is presented. Analyses were performed to develop accident surrogate measures for hazardous location identification and countermeasures evaluation at rural isolated curves on two-lane roads, rural signalized intersections, and two-lane tangent sections in urban areas.

The reports may be purchased from NTIS.

Review of Channelizing Devices for Two-Lane Two-Way Operations, Report No. FHWA/RD–83/056, and Performance Criteria for Channelizing Devices Used for Two-Lane Two-Way Operations, Report No. FHWA/RD–83/057

by Traffic Control and Operations Division

When one directional roadway of a divided, four-lane highway is closed for construction or maintenance, two-lane, two-way operation (TLTWO) is created when the traffic that normally uses that roadway is crossed over the median and shares the other roadway with opposing traffic. Traffic on this open roadway operates one lane in each direction with passing prohibited.

Report No. FHWA/RD–83/056 examines the use, maintenance practices, and cost factors for existing TLTWO channelizing devices. Modified and new device concepts are identified and evaluated. Two channelizing device concepts were selected for further study. Report No. FHWA/RD–83/057 describes the development of functional requirements for these two devices—an improved flexible, durable, self-restoring device of modest size having a post or paddle shape and a continuous, raised median with intermittent, high-visibility devices mounted on it. Performance criteria and testing procedures for these two TLTWO channelizing devices are described.

The reports may be purchased from NTIS.

Pavement Friction Measurements on Nontangent Sections of Roadways, Vol. I, Summary Report, and Vol. II, Comprehensive Report, Report Nos. FHWA/RD–82/149–150

by Pavement Division

These reports discuss a study to develop the most practical system for measuring wet-pavement friction on nontangent road sections. The measurement system must be capable of operating at traffic speeds, should not be overly complex, should have its own watering and data collection systems, and must be compatible and correlate with surface friction testers currently used.

It was found that friction on nontangent sections can be measured with an ASTM E–274 skid trailer when both the dynamic longitudinal and vertical forces on the test wheel are determined during locked-wheel tests. The limit of performance of the E–274 system is ±0.35 g lateral acceleration in the horizontal plane of the test trailer. On nontangent tests, dynamic vertical forces should be determined either by a load cell or from accelerations acting on the trailer.

The reports may be purchased from NTIS.

Prestressed Pavement, Vols. 1–3, Report Nos. FHWA/RD–82/090–092

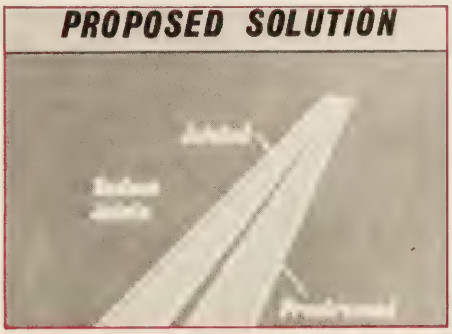
by Pavement Division

In designing prestressed pavement, pavement thickness must be determined and joint hardware selected. The process involves the interaction of many factors.

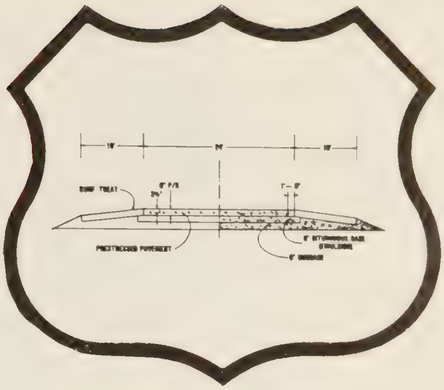
Conventional concrete pavements are designed on the basis of concrete's relatively low modulus of rupture without effectively using the natural advantage of its high compressive strength. Prestressing concrete decreases the tensile stress in the flexural zone, which reduces or eliminates pavement cracking, decreases the number of transverse joints, provides a more comfortable riding surface, and reduces maintenance costs.

Volume 1, **Joint Designs**, presents four transverse joint designs for prestressed concrete pavements. Volume 2, **Thickness Design**, presents a computerized procedure for the thickness design of "zero maintenance" prestressed pavements. Input data include traffic loading, temperature and moisture variations in concrete slab, loss of subbase support, properties of concrete, subbase and subgrade, and effective mid-slab prestress. The procedure is based on a flexural stress analysis at the bottom longitudinal edge of the slab.

Volume 3, **Construction Manual**, presents procedures and materials for prestressed pavement construction. Details of bulkhead construction, anchorage assemblies, jacking accessories, and joint hardware placement are illustrated.



The reports may be purchased from NTIS.



Prestressed Pavement Performance in Four States, Report No. FHWA/RD–82/169

by Pavement Division

This report discusses the performance of four full-scale prestressed concrete pavements built between 1971 and 1979. The projects were constructed in different climatic zones (Virginia, Pennsylvania, Mississippi, and Arizona) and carry a range of traffic.

A panel of representatives from the four States inspected the condition of these projects in March 1981 and concluded that the prestressed concrete pavements had performed adequately. On the basis of performance and economics, prestressed pavement is a viable alternative. Some design guidelines and desirable future project characteristics are suggested in the report.

The report may be purchased from NTIS.

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

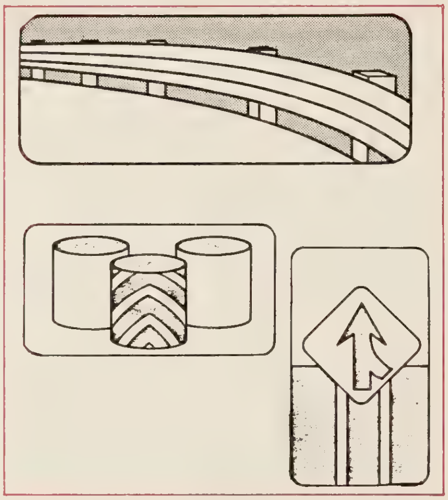
The following are brief descriptions of selected items that have been completed recently by State and Federal highway units in cooperation with the Office of Implementation, Offices of Research, Development, and Technology, Federal Highway Administration. Some items by others are included when they have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

Federal Highway Administration
Office of Implementation, HRT–1
6300 Georgetown Pike
McLean, Virginia 22101

A Procedure for Determining Frequencies for Inspection and Repair of Highway Safety Hardware, Report No. FHWA–IP–83–4

by Office of Implementation

This manual describes a procedure that considers the expected frequency of motor vehicles hitting highway safety hardware and the consequences of the hits to the hardware to determine optimum times to inspect and repair the hardware. The five-step procedure includes obtaining frequency data on traffic accidents involving specific items of highway hardware, ranking accident locations where highway hardware items are involved, sorting the locations by road class and identifying the accident groups,



identifying the range of intervals for each group, and selecting a level of service. An appendix to the manual includes a step-by-step illustration of the new procedure.

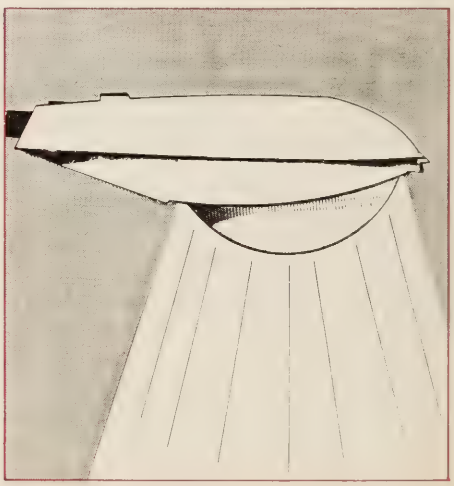
The manual may be purchased from NTIS.

Lighting Handbook, Designing the Lighting System—Using Pavement Luminance, Addendum to Chapter 6 of the Roadway Lighting Handbook

by Office of Implementation

This report supplements and updates the Roadway Lighting Handbook (Report No. FHWA–IP–78–15), which deals with all aspects of visibility requirements, equipment, system design, operation, and maintenance of a roadway lighting system. A major change from illuminance to pavement luminance as criteria for lighting design is explained in this report, which is an addendum to chapter 6 on lighting design in the Handbook. The meaning of the new terminology is explained, luminance criteria are given, and design procedures are discussed. Alternate methods of design using either a hand calculator, graphical aids, or computer printouts are included. This change in lighting design criteria should be more energy efficient and provide easier and faster discernment of driving tasks at night.

The report may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–00271–0). The Roadway Lighting Handbook also may be purchased (Stock No. 050–003–00339–5).



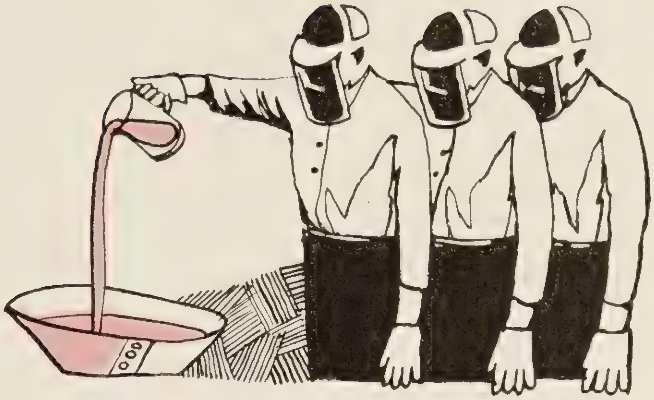
Restructured FCP

The Federally Coordinated Program (FCP) of Highway Research, Development, and Technology was established in 1971 to coordinate and manage Federal and State research and development activities. The structure of the FCP provides a logical framework for classifying the major program areas and distinguishing the subelements of these areas, for assigning budget resources and responsibilities, for determining objectives, and for communicating the program to the FHWA field offices and States. The FCP consists of broad categories containing carefully selected projects that concentrate available resources on obtaining timely solutions to urgent national highway problems.

The FCP was revised for Fiscal Year 1984. The new simplified structure is more responsive to FHWA and State priorities and reflects the 1982 reorganization of FHWA's Offices of Research, Development, and Technology. The restructuring of the program also reflects the transition of the U.S. highway program from major interstate highway construction to restoration and reconstruction of the existing system.

In restructuring the FCP, some projects have been consolidated into a new project, others have been divided with portions transferred to new projects or categories, and some projects remain intact.

Further information on the FCP is found in the **1983 Federally Coordinated Program of Highway Research, Development, and Technology**. Copies of this annual report will be available in April and can be obtained from the Office of Operations Staff, HRD–10, Federal Highway Administration, 6300 Georgetown Pike, McLean, Virginia 22101.



New Research in Progress

The following new research studies reported by FHWA's Offices of Research, Development, and Technology are sponsored in whole or in part with Federal highway funds. For further details on a particular study, please note the kind of study at the end of each description and contact the following: Staff and administrative contract research—*Public Roads* magazine; Highway Planning and Research (HP&R)—performing State highway or transportation department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Highway Design and Operation for Safety

FCP Project 1A: Traffic and Safety Control Devices

Title: Effects of Shoulder Texture Treatments on Safety. (FCP No. 31A3084)
Objective: Determine if a comparison of rough textured (noisy) shoulders versus smooth shoulders on high-volume rural highways (freeways and expressways) indicates a significant reduction in the number of single-vehicle run-off-the-road accidents. If so, determine the locations where such treatments are cost-effective. Examine various textured treatments on rural roadways to evaluate their effects on drivers straying from the traveled way. Consider alternative concepts, such as various percent of textured treatment width per shoulder width.
Performing Organization: AMAF Industries, Inc., Columbia, Md. 21045
Expected Completion Date: December 1984
Estimated Cost: $157,000 (FHWA Administrative Contract)

FCP Project 1K: Accident and Countermeasure Analysis

Title: Inexpensive Accident Countermeasures at Narrow Bridges. (FCP No. 31K2162)
Objective: Examine the effectiveness of inexpensive accident countermeasures, such as delineation and signing, in reducing narrow bridge accidents. Use accident surrogates such as vehicle lateral placement and speed profiles as measures of effectiveness.
Performing Organization: Goodell-Grivas, Inc., Southfield, Mich. 48075
Expected Completion Date: September 1986
Estimated Cost: $226,830 (FHWA Administrative Contract)

FCP Project 1M: Rural Two-Lane Highways

Title: Cost-Effective Cross Section Design for Two-Lane Rural Roads. (FCP No. 31M2693)
Objective: Determine the benefits and costs of various combinations of lane widths, shoulder widths, and shoulder surface types for two-lane rural roads carrying less than 4,000 vehicles per day. Develop national estimates of the costs and benefits that might be achieved by developing the two-lane rural road network to different combinations of lane widths, shoulder widths, and shoulder surface types.
Performing Organization: Goodell-Grivas, Inc., Southfield, Mich. 48075
Expected Completion Date: September 1985
Estimated Cost: $203,840 (FHWA Administrative Contract)

FCP Project IR: Speed Zoning and Control

Title: Guidelines for Establishing Speed Zones. (FCP No. 31R1024)
Objective: Obtain current criteria used in establishing speed zones from a sample of State and local highway agencies. Conduct speed studies on a nationwide sample of streets and highways to determine if existing speed limits (including maximum, minimum, and advisory speed limits) are reasonable. Identify road, vehicle, and driver factors significantly affecting speed and accident risk. Validate the 85th percentile speed, and refine criteria to eliminate the subjectiveness in considering the factors affecting proper speed limits.
Performing Organization: Martin R. Parker & Associates, Inc., Canton, Mich. 48187
Expected Completion Date: October 1985
Estimated Cost: $241,850 (FHWA Administrative Contract)

FCP Project IV: Roadside Safety Hardware for Nonfreeway Facilities

Title: Effects of Changes in Effective Rail Height on Barrier Performance. (FCP No. 31V1032)
Objective: Investigate the effects of rail height variations attributed to construction practices, resurfacing, settlement, erosion, and soil and turf buildup on the safety performance of various roadside barriers. Use computer simulation and full-scale testing.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: December 1985
Estimated Cost: $320,200 (FHWA Administrative Contract)

Title: Impact Attenuators—A Current Engineering Evaluation. (FCP No. 31V3022)
Objective: Develop the relationship between vehicle crash parameters and occupant injury. Conduct a full-scale crash test to provide insight into problems associated with frozen sand in impact attenuators. Investigate the interactions of minisized vehicles colliding with impact attenuators currently used on U.S. highways.
Performing Organization: ENSCO, Inc., Springfield, Va. 22151
Expected Completion Date: August 1985
Estimated Cost: $297,660 (FHWA Administrative Contract)

FCP Project 1W: Vehicle/Surface Interaction Problems

Title: Methodology for Road Roughness Profiling and Rut Depth Measurement. (FCP No. 31W3062)
Objective: Install noncontact height sensors in a survey vehicle to collect pavement surface data (roughness profiles and rut depth in one wheel track and cross slopes and grades) at regular traffic speeds. Develop several optional configurations to be selected by potential users. Develop a complete, operational prototype of the system and procedures for its use.
Performing Organization: University of Michigan, Ann Arbor, Mich. 48109
Expected Completion Date: March 1986
Estimated Cost: $284,260 (FHWA Administrative Contract)

Title: Procedures for the Analysis of Pavement Condition Data. (FCP No. 31W4022)
Objective: Develop procedures for processing and analyzing pavement condition data using existing computer programs. Make these data compatible with pavement management systems information needs. State highway departments will supply survey data and assist in evaluating the procedures. Develop and present a workshop to demonstrate and instruct in the use of the procedures.
Performing Organization: ARE, Inc., Austin, Tex. 78746
Expected Completion Date: March 1986
Estimated Cost: $284,900 (FHWA Administrative Contract)

FCP Project 1Z: Implementation of Safety Research and Development

Title: Traffic Signal Design Course. (FCP No. 31Z1013)
Objective: Develop a 4-day intensive training course that will enable practitioners to apply modern traffic engineering techniques when designing signals. Develop a 1 1/2-day course that will improve traffic signal design skills. Apply modern value engineering techniques to the design traffic signals. Apply modern maintenance management techniques as related to the design, installation, and operation of traffic signals.
Performing Organization: JHK and Associates, San Francisco, Calif. 94119
Expected Completion Date: December 1984
Estimated Cost: $342,880 (FHWA Administrative Contract)

Title: User-Friendly Texas Model for Intersection Traffic. (FCP No. 41ZA018)
Objective: Create a user-friendly input processor and a user-usable set of graphics outputs for the "Texas" model. Reprogram the "Texas" model into Fortran 77.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: September 1985
Estimated Cost: $200,800 (HP&R)

FCP Category 2—Traffic Control and Management

FCP Project 2L: Electronic Devices for Traffic Control

Title: Alternative Incandescent Traffic Signal Lamps and Systems for Improved Optical and Energy Efficiency. (FCP No. 32L2152)
Objective: Evaluate alternative traffic signal components (lamps, reflectors, and lenses) for improving the optical and energy efficiency of the traffic signal. Identify methods and benefits of night dimming of the traffic signals, including analysis of projected costs, estimated savings, and implementability of each option.
Performing Organization: Lighting Sciences, Inc., Scottsdale, Ariz. 85260
Expected Completion Date: September 1984
Estimated Cost: $121,970 (FHWA Administrative Contract)

FCP Project 2Q: Urban Network Control

Title: Arterial Analysis Package—Maintenance and Support. (FCP No. 32Q9168)
Objective: Make the arterial analysis package and its component models easy, convenient, and inexpensive to implement and use, reliable and useful to the traffic engineering community, and functional under a portable system.
Performing Organization: University of Florida, Gainesville, Fla. 32611
Expected Completion Date: September 1986
Estimated Cost: $178,270 (FHWA Administrative Contract)

FCP Category 4—Pavement Design, Construction, and Management

FCP Project 4B : Design and Rehabilitation of Rigid Pavements

Title: Purchase and Adapt a Falling Weight Deflectometer for Nondestructive Evaluation and Research on Rigid Pavements in Texas. (FCP No. 44B1032)
Objective: Purchase a Dynatest falling weight deflectometer and compare it with a Dynoflect and the spectral analysis of surface waves method (SASW). Use the falling weight deflectometer to evaluate nonlinear stress sensitivity, load transfer, and void determinations. Simulate the transient loading in addition to the static modeling of elastic layer theory using a dynamic analysis model with the improved falling weight deflectometer. Evaluate actual rigid pavement job sites.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: September 1985
Estimated Cost: $131,000 (HP&R)

Title: Condition Surveys and Performance Monitoring of Existing and Overlaid Rigid Pavements. (FCP No. 44B1042)
Objective: Conduct another Statewide condition survey including overlaid sections. Modify Program PRPO1 to include overlaid pavements in the prioritization scheme. Check performance models against actual performance to evaluate subbase type, construction technique, construction season, traffic, swelling clay loss factors, and coarse aggregate type. Use accumulated condition data to verify the RPOD2 and RPRDS1 computer design programs. Collect various cost information to be used in the decisionmaking process.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas State Department of Highways and Public Transportation
Expected Completion Date: September 1985
Estimated Cost: $116,200 (HP&R)

FCP Project 4C: Design and Rehabilitation of Flexible Pavements

Title: Effects of Asphalt Composition and Compaction on the Performance of Asphalt Pavement Mixtures. (FCP No. 44C6064)
Objective: Improve the performance of asphalt pavements, particularly the thin surface courses, by examining composition of the asphalt binder and the compaction procedure. Measure the effects of composition and compaction procedure by resistance of the mix to accelerated deterioration.

Performing Organization: Purdue University, West Lafayette, Ind. 47907
Funding Agency: Indiana Department of Highways
Expected Completion Date: August 1986
Estimated Cost: $97,450 (HP&R)

FCP Project 4D: Improved Flexible Binders

Title: Properties of Asphalt Cements Used in Virginia and Their Effects on Pavement Performance. (FCP No. 44D1082)
Objective: Determine the characteristics of asphalts used in pavement construction in Virginia during 1983 and compare them with those of asphalts (mostly from Venezuelan or Middle East crudes that formed the basis of presently used design procedures) available throughout the Eastern United States before the 1973 oil embargo. Summarize the literature concerning chemical composition of asphalt to evaluate the significance of any noted changes. Emphasize the relationship of chemical composition of asphalts and their physical characteristics and performance.
Performing Organization: Virginia Department of Highways and Transportation, Richmond, Va. 23219
Expected Completion Date: March 1985
Estimated Cost: $134,290 (HP&R)

FCP Project 4K: Cost-Effective Rigid Concrete Construction and Rehabilitation in Adverse Environments

Title: In Situ Measurement of Rate of Corrosion of Reinforcing Steel in Concrete. (FCP No. 34K1012)
Objective: Evaluate the suitability of the rate of corrosion measuring device for measuring in situ rate of corrosion of steel in concrete. Improve and modify the instrument as necessary and make it fully automatic so that button operation will activate the instrument to polarize, measure the corrosion rate, and analyze and display the results.
Performing Organization: National Bureau of Standards, Washington, D.C. 20234
Expected Completion Date: September 1985
Estimated Cost: $150,000 (FHWA Administrative Contract)

Title: Cathodic Protection for Prestressed Systems. (FCP No. 34K3017)
Objective: Selectively sample and analyze bridges that incorporate pre- or post-tensioning systems to determine if active corrosion is taking place at the prestressing steel. Examine current cathodic protection systems in light of potential applications to prestressed systems. Identify limitations and potential problems. Modify existing technology on cathodic protection as necessary to adapt to prestressed systems. Conduct pilot laboratory studies and recommend field applications for successful cathodic protection systems.

Performing Organization: HARCO Corporation, Medina, Ohio 44256
Expected Completion Date: September 1986
Estimated Cost: $150,000 (FHWA Administrative Contract)

Title: Protective Systems for New Prestressed and Substructure Concrete. (FCP No. 34K3018)
Objective: Define, test, and evaluate protective systems for new conventionally reinforced and prestressed concrete bridge members constructed in an adverse salt environment. Emphasize structures located in the marine environment, but protective systems developed should be applicable to structures exposed to deicing chemicals.
Performing Organization: Wiss, Janney, Elstner Associates, Northbrook, Ill. 60062
Expected Completion Date: October 1986
Estimated Cost: $176,000 (FHWA Administrative Contract)

FCP Category 5—Structural Design and Hydraulics

FCP Project 5H: Highway Drainage and Flood Protection

Title: Test and Evaluation of Expressway Drainage Design. (FCP No. 35H2092)
Objective: Develop a complete data collection package for collecting field expressway runoff data in the evaluation of expressway drainage system modeling.
Performing Organization: U.S. Geological Survey, Reston, Va. 22092
Expected Completion Date: December 1984
Estimated Cost: $119,500 (FHWA Administrative Contract)

FCP Project 5M: Low Volume Roads

Title: Design and Operation of Aggregate-Surfaced Roads. (FCP No. 35M2062)
Objective: Develop representative designs of aggregate-surfaced roads for various climates, with corresponding performance history and maintenance for various levels of traffic. Evaluate methods

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to integrate subgrade properties into the design procedure. Use the collected information, along with additional data from literature reviews, to develop a design procedure (including the total roadway as well as the aggregate surfacing) with up to three optional levels of engineering and material requirements, depending on traffic loads and frequencies.

Performing Organization: Michigan Technological University, Houghton, Mich. 49931
Expected Completion Date: September 1985
Estimated Cost: $168,850 (FHWA Administrative Contract)

FCP Project 5N: Pavement Management Strategies

Title: Impact of Pavement Maintenance on Damage Rate. (FCP No. 35N3072)
Objective: Develop a handbook to evaluate the impacts of maintenance and rehabilitation on damaged composite, asphalt, and jointed and continuously reinforced concrete pavements.
Performing Organization: Austin Research Engineers, Inc., Austin, Tex. 78746
Expected Completion Date: September 1985
Estimated Cost: $195,200 (FHWA Administrative Contract)

FCP Category 6—Improved Technology for Highway Construction

FCP Project 6D: Structural Rehabilitation of Pavement Systems

Title: Pressure Relief and Other Joint Rehabilitation Techniques. (FCP No. 36D2864)
Objective: Implement a variety of design concepts in existing pavements for further development and evaluation to provide valuable insight for potential improvements in joint designs that can be incorporated in the construction of new pavements.
Performing Organization: ERES Consultants, Inc., Champaign, Ill. 61820
Expected Completion Date: September 1986
Estimated Cost: $233,490 (FHWA Administrative Contract)

FCP Category 7—Improved Technology for Highway Maintenance

FCP Project 7A: Improved Highway Maintenance Practices

Title: Improved Methods for Patching on High-Volume Roads. (FCP No. 37A1022)
Objective: Identify and evaluate standard, new, and improved methods for permanent patching of high-volume bituminous and portland cement concrete pavements. Identify methods presently used by highway agencies for repairing high-volume roads, determine which methods are best for various applications, offer possible suggestions for improving the methods, and field evaluate the new methods and their improved versions.

Performing Organization: Byrd, Tallamy, MacDonald & Lewis, Falls Church, Va. 22042
Expected Completion Date: December 1984
Estimated Cost: $115,480 (FHWA Administrative Contract)

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